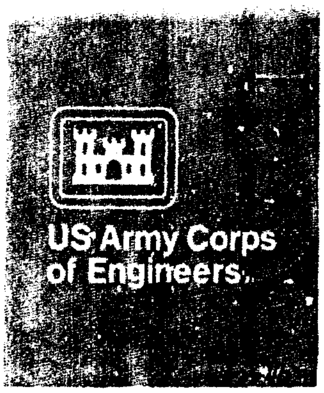

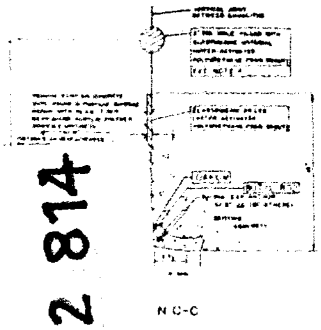

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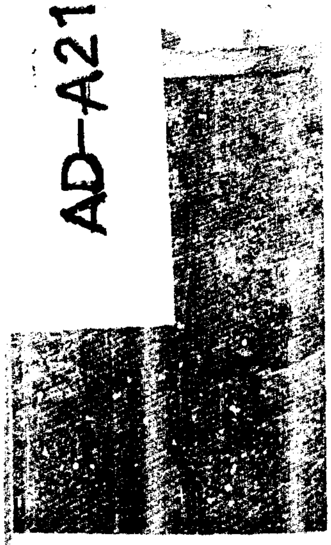

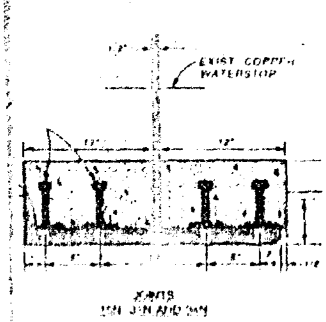

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REPAIR, EVALUATION, MAINTENANCE, AND REHABILITATION RESEARCH PROGRAM

TECHNICAL REPORT REMR-CS-22

MONOLITH JOINT REPAIRS: CASE HISTORIES

by

James G. May, James E. McDonald

Structures Laboratory

DEPARTMENT OF THE ARMY
Waterways Experiment Station, Corps of Engineers
PO Box 631, Vicksburg, Mississippi 39181-0631

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The following two letters used as part of the number designating technical reports of research published under the Repair, Evaluation, Maintenance, and Rehabilitation (REMR) Research Program identify the problem area under which the report was prepared:

	Problem Area		Problem Area
CS	Concrete and Steel Structures	EM	Electrical and Mechanical
GT	Geotechnical	EI	Environmental Impacts
HY	Hydraulics	OM	Operations Management
CO	Coastal		

COVER PHOTOS:

TOP — Repair details, Lock and Dam No. 2, Mississippi River.

CENTER — Example of monolith joint deterioration, Bayou Sorrel Lock.

BOTTOM — Monolith joint repair details for joints 15N, 31N, and 34N, Algiers Lock.

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19. ABSTRACT (Continue on reverse if necessary and identify by block number)

The Corps of Engineers currently operates and maintains 545 dams and 269 lock chambers at 605 sites. These structures are routinely exposed to deleterious forces, such as impact and abrasion damage from navigation traffic. In addition, nearly one-half of the 269 lock chambers were built prior to 1940 or before concrete was intentionally air-entrained. Seventy-eight percent of these nonair-entrained locks are located in the Corps' North Central and Ohio River Divisions where they are subjected to many cycles of freezing and thawing.

Monolith joints are the fourth most common location for deficiencies in dams and the third for locks. The objective of this study was to identify materials and techniques used to repair deteriorated monolith joints, excluding joint sealant failures or seepage. A secondary objective was the identification of areas in which further research is needed to supplement existing technology.

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19. ABSTRACT (Continued).

Information on the repair of monolith joints for the seven case histories was obtained through (a) review of the US Army Engineer Waterways Experiment Station damage and repair data base for Corps Civil Works Structures, (b) review of periodic inspection reports, (c) visits to project sites, and (d) discussion with project personnel.

Although the information obtained from the various sources varied widely from project to project, attempts were made to obtain (a) a description of the project, (b) the cause and extent of monolith joint deterioration, (c) descriptions of repair materials and procedures, and (d) performance to date of the repaired monolith joints.

PREFACE

The study reported herein was authorized by Headquarters, US Army Corps of Engineers (HQUSACE), under Civil Works Research Work Unit 32307, "Techniques for Joint Repair and Rehabilitation," for which MAJ James G. May, CE, is the Principal Investigator. This work unit is a part of the Concrete and Steel Structures Problem Area of the Repair, Evaluation, Maintenance, and Rehabilitation (REMR) Research Program sponsored by HQUSACE. The Overview Committee at HQUSACE for the REMR Research Program consists of Mr. James E. Crews and Dr. Tony C. Liu. Technical Monitor for this study was Dr. Liu.

This study was performed at the US Army Engineer Waterways Experiment Station (WES) under the general supervision of Mr. Bryant Mather, Chief, Structures Laboratory (SL), and Mr. Kenneth L. Saucier, Chief, Concrete Technology Division (CTD), and under the direct supervision of Mr. James E. McDonald, Problem Area Leader for the Concrete and Steel Structures Problem Area. The report was edited by Ms. Gilda F. Miller, and text and figure layout was coordinated by Ms. Chris Habeeb, Information Products Division, Information Technology Laboratory, WES. Program Manager for the REMR Research Program is Mr. William F. McCleese, CTD.

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Mr. Steve Tatro, Walla Walla
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Acting Commander and Director of WES during publication of this report was LTC Jack R. Stephens, EN. Technical Director was Dr. Robert W. Whalin.

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CONVERSION FACTORS, NON-SI TO SI (METRIC)
UNITS OF MEASUREMENT

Non-SI units of measurement used in this report can be converted to SI (metric) units as follows:

Multiply	By	To Obtain
acres	4,046.873	square metres
acre-feet	1,233.489	cubic metres
cubic feet	0.02831685	cubic metres
degrees (angle)	0.01745329	radians
Fahrenheit degrees	5/9	Celsius degrees or kelvins*
feet	0.3048	metres
gallons (US liquid)	3.785412	cubic metres
quarts (US liquid)	0.9463529	litres
inches	25.4	millimetres
miles (US statute)	1.609347	kilometres
ounces (avoirdupois)	0.02834952	kilograms
pounds (force) per square inch	0.006894757	megapascals
pounds (mass)	0.4535924	kilograms
pounds (mass) per cubic yard	0.5932764	kilograms per cubic metre
tons (2,000 pounds, mass)	907.1847	kilograms

* To obtain Celsius (C) temperature readings from Fahrenheit (F) readings, use the following formula: $C = (5/9)(F - 32)$. To obtain kelvin (K) readings, use: $K = (5/9)(F - 32) + 273.15$.

MONOLITH JOINT REPAIRS: CASE HISTORIES

PART I: INTRODUCTION

Background

1. The Corps of Engineers currently operates and maintains 545 dams and 269 lock chambers at 605 sites. These structures are routinely exposed to deleterious forces, such as impact and abrasion damage from navigation traffic. In addition, nearly one-half of the 269 lock chambers were built prior to 1940 or before concrete was intentionally air-entrained. Seventy-eight percent of these nonair entrained locks are located in the Corps' North Central and Ohio River Divisions where they are subjected to many cycles of freezing and thawing.

2. A review of the US Army Engineer Waterways Experiment Station (WES) damage and repair data base (McDonald and Campbell 1985) indicated that monolith joints are the fourth most common location for deficiencies in dams and the third for locks. These deficiencies are related to joint sealant failures or seepage through the monolith joints 48 percent of the time for dams and 46 percent for locks. McDonald (1986) and Campbell and Bean (1988) address these types of deficiencies. Spalling, the second most frequently reported deficiency at monolith joints, occurs 33 percent and 39 percent of the time in dams and locks, respectively.

Purpose

3. The objective of this study was to identify materials and techniques used to repair monolith joint deficiencies other than joint sealant failures or seepage. A secondary objective was the identification of areas in which further research is needed to supplement existing technology.

Scope

4. Information on the repair of monolith joints was obtained through (a) review of the WES damage and repair data base for Corps Civil Works structures, (b) review of periodic inspection reports, (c) visits to project

sites, and (d) discussion with project personnel.

5. Although the information obtained from the various sources varied widely from project to project, attempts were made to obtain (a) a description of the project, (b) the cause and extent of monolith joint deterioration, (c) descriptions of repair materials and procedures, and (d) performance to date of the repaired monolith joints.

PART II: CASE HISTORIES

6. Sufficient information was obtained to prepare a case history for monolith joint repairs at seven different projects. Descriptions of the repairs are arranged in chronological order in the following paragraphs.

Lower Monumental Lock and Dam

7. Lower Monumental Lock and Dam is located in southeastern Washington on the Snake River, 41 miles* above the confluence with the Columbia River. It consists of a six-bay powerhouse, two nonoverflow sections, spillway and navigation lock chamber (Figure 1). The dam is 3,800 ft long, and the spillway is 143 ft high. The navigation lock chamber is 86 ft wide by 675 ft long and has a maximum lift of 103 ft. The upstream submergible lift gate has an effective height of 20 ft, and the downstream lift gate has an effective height of 83 ft (Figure 2). Construction began in April 1961, and the pool was raised in February 1969. During construction, workability, water-cement ratio, and slump problems were experienced with the lock wall concrete. These problems were attributed to a high percentage of soft particles in the fine aggregate as determined by US Army Engineer District (USAED), Walla Walla (1969).

8. Initial observations of scribe marks and electrolevels on monoliths 25, 26, 27, and 28 indicated that monoliths 27 and 28 had a two-component movement. This movement subjected the water stops to exceptional strains. When the lock was filled, the joint between monoliths 25 and 27 opened approximately 0.04 in.; the joint between monoliths 26 and 28 opened approximately 0.06 in. During the first periodic inspection (USAED, Walla Walla, 1969), the joint between monoliths 26 and 28 began to leak when the lock was filled. Shortly thereafter, the joint between monoliths 25 and 27 began to leak. This condition, which increased the load on the monoliths beyond the load considered in the design stability calculations, resulted in failure of the water stops and the deterioration of the concrete adjacent to the joints.

9. The failed water stops between monoliths 26 and 28 and 25 and 27 were repaired by contract during dewatering in November 1970. Two coats of an

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* A table of factors for converting non-SI units of measurement to SI (metric) units is presented on page 3.

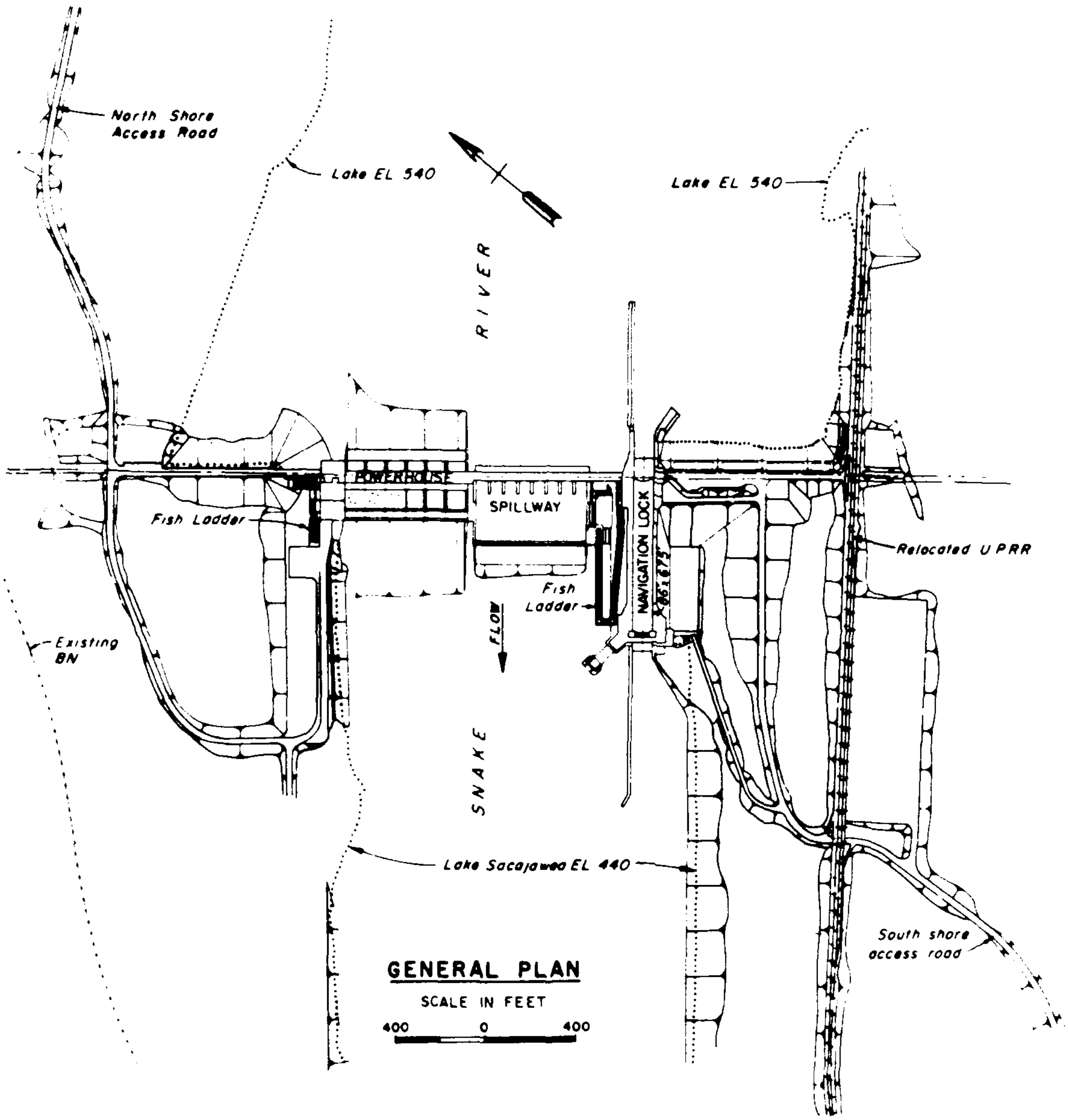

Figure 1. General plan, Lower Monumental Lock and Dam

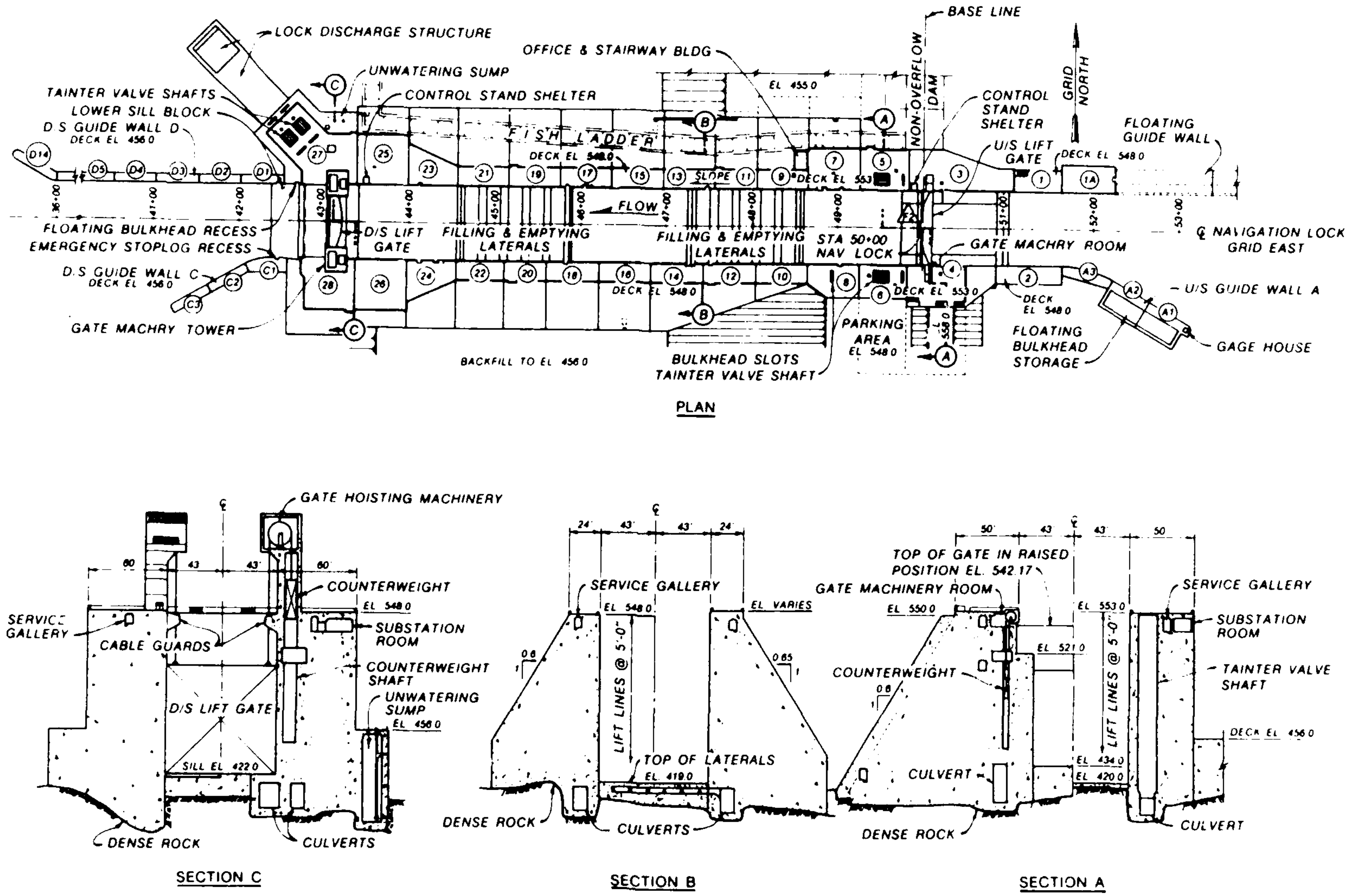

Figure 2. Plan and typical sections, Lower Monumental Lock

epoxy, Colma-Kote M. manufactured by Sika Corporation, were applied over the deteriorated concrete to form a membrane. Large wedges of concrete on the inside face of the lock at deck level on both joints were removed and replaced in one repair operation.

10. Periodic Inspection Reports No. 4 (USAED, Walla Walla, 1973) and No. 5 (USAED, Walla Walla, 1975), documented the continued deterioration of the concrete adjacent to these joints. The deterioration extended from el 420* to 460 on monolith 26 and el 480 to 537 on monolith 28. The cause of the deterioration (Figure 3) was described in Periodic Inspection Report No. 5 (USAED, Walla Walla, 1975) as follows:

> Damage is caused by relative movements of adjacent monoliths from hydraulic loading during lockages. These movements are much more significant during the cold winter months when the concrete has contracted and there is less friction along the monolith joint. When the locks are emptied the upstream monoliths 25 and 26 move inward relative to the larger downstream monoliths 27 and 28, thereby causing a tearing action along the joint surface. This action spalls the concrete through repeated shearing and continued crack propagations.

11. In March 1975, the interior faces of monolith joints 25 and 27 and 26 and 28 were repaired with a fibrous concrete pour-back. Design Memorandum No. 9 Navigation Facilities Supplement No. 2, "Monolith and Joint Repairs" (USAED, Walla Walla, 1977b), provided the following description of the repair technique used in 1975:

> The procedure consists of removing all loose concrete, saw cutting around the perimeter of damaged area that is not at least 15 inches deep, grouting horizontal anchors into the monolith (these anchors will serve as anchors for the pour-back repair concrete and for form supports), establishing a free joint between the repair concrete and adjacent monolith by installing a compressible joint board and filling the repair area with fibrous concrete from the bottom to the top in one continuous placement. Form pressures will stress the anchors which then hold the repair in place after it sets. A prestressed surface patch is thereby created. By using fibrous concrete, rebar is not required and the material will be more resistant than conventional concrete to impact, shear, and other forces.

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* All elevations (el) cited herein are in feet referred to the National Geodetic Vertical Datum (NGVD).

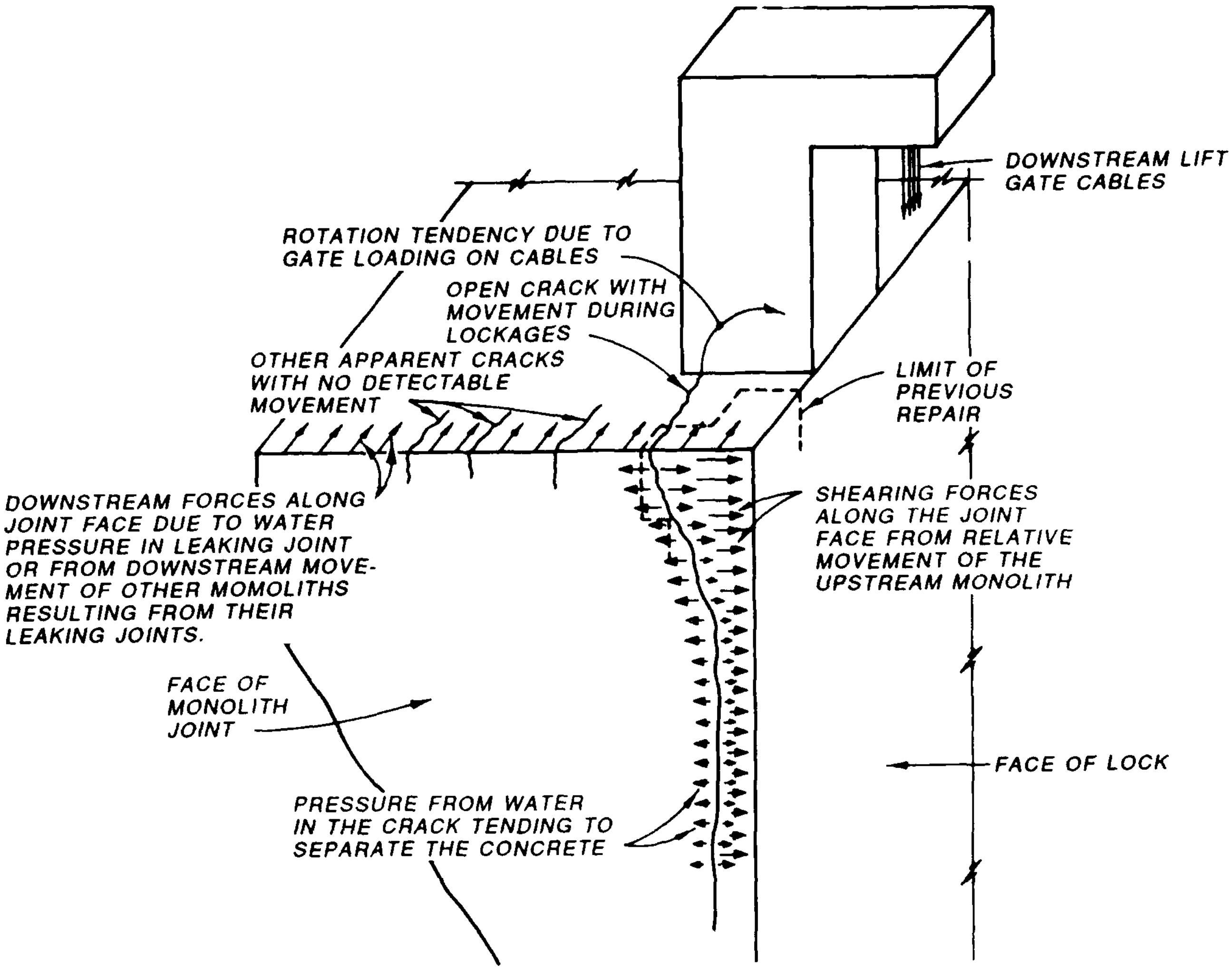


Figure 3. Forces acting on gate monolith, Lower Monumental Lock and Dam

Figure 4 illustrates the technique described (USAED, Walla Walla, 1977b). Strain gages used during the placement of the fibrous concrete indicated that high rates of placement caused little increase in form pressure, regardless of lift height. Also, the original design had called for the monoliths to join tightly without a joint board. Wherever this type of joining occurred, spalling invariably resulted. Conversely, wherever joint boards were installed, spalling was not a problem.

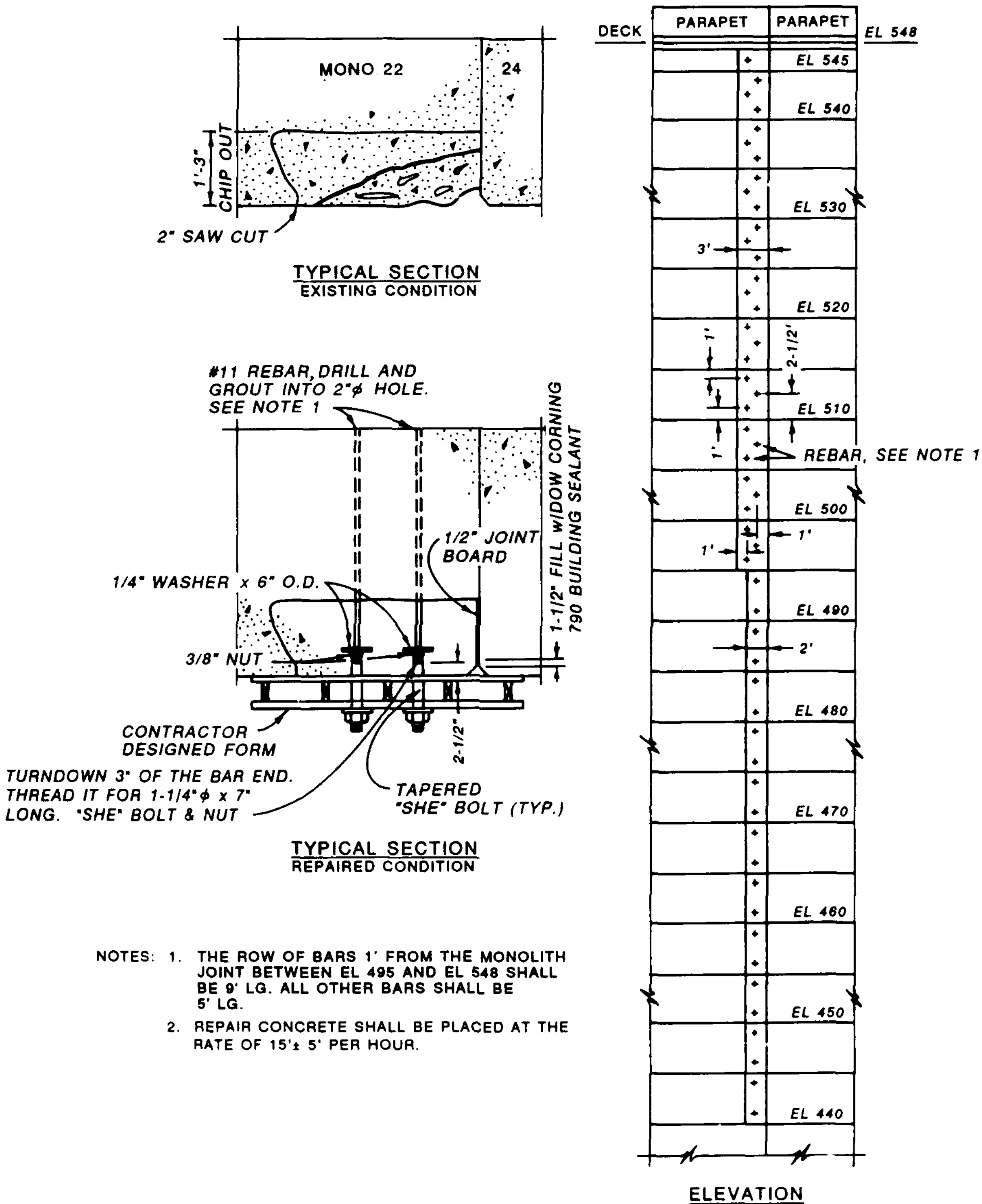
12. Subsequent inspection reports (USAED, Walla Walla, 1977a, 1981, and 1982) indicate that the fibrous-concrete pour-back repair at joints 25 and 27 and 26 and 28 continued to perform well. This technique was also used to repair similar spalling which occurred on the exterior face of monolith joints 26 and 28. Figures 5 through 9 illustrate the sequence of deterioration and repair on the exterior face of monolith joints 26 and 28. The fibrous-concrete pour-back repairs continue to perform well to date.

Dresden Island Lock and Dam

13. The Dresden Island Lock and Dam is located immediately downstream of the confluence of the Des Plaines and Kankakee Rivers at mile 271.5 of the Illinois Waterway near Morris, IL. The lock has a usable chamber of 110 by 600 ft with miter gates at both ends. Normal lift is 21.75 ft. The lock walls are concrete gravity type founded on rock (Figure 10). The upper guide wall consists of freestanding concrete piers in the upper pool with a concrete beam at the top of the piers. The lower guide wall is a concrete gravity wall which retains backfill from the land side. The upper miter gate sills are concrete arches and the lower sill is a thin concrete paving over the foundation rock. The dam includes an overflow spillway, tainter gates, ice chute, head gates, and a concrete arch. The project was completed in 1933 at a cost of $3,915,964 (McDonald 1987).

14. In 1954 the upstream half of the lock chamber was resurfaced with shotcrete. Resurfacing extended over the even numbered monoliths 12 through 20 for the land wall and over the odd numbered monoliths 11 through 29 for the river wall (Figure 11). This repair was described in detail in Technical Report REMR-CS-13 (McDonald 1987).

15. During the third periodic inspection (USAED, Chicago, 1978), it was noted that the shotcrete was in relatively good condition except at the top of



NOTES: 1. THE ROW OF BARS 1' FROM THE MONOLITH JOINT BETWEEN EL 495 AND EL 548 SHALL BE 9' LG. ALL OTHER BARS SHALL BE 5' LG.

2. REPAIR CONCRETE SHALL BE PLACED AT THE RATE OF 15'± 5' PER HOUR.

Figure 4. Monolith joint repair details, Lower Monumental Lock and Dam

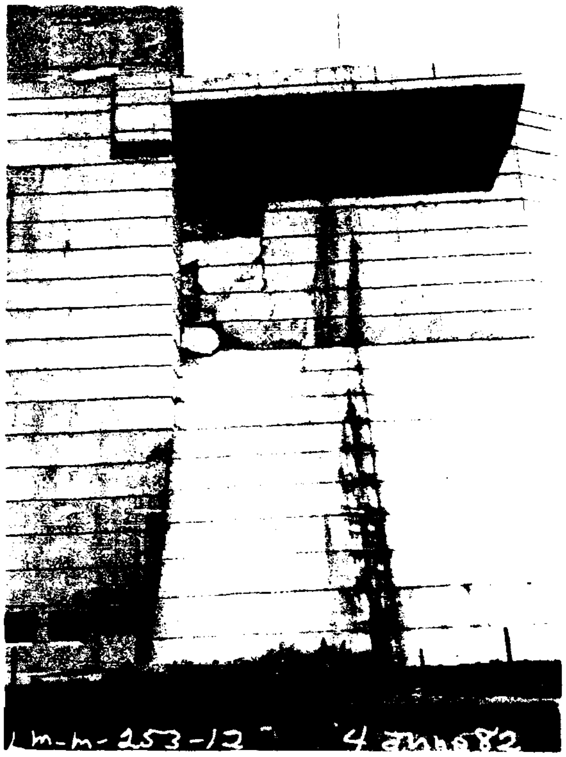


Figure 5. Monolith joint deterioration, 4 June 1982, Lower Monumental Lock and Dam

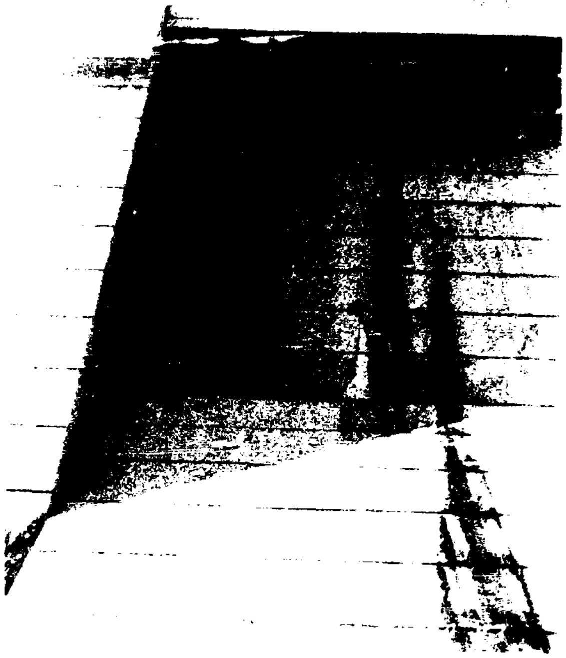


Figure 6. Monolith joint deterioration, 18 March 1983, Lower Monumental Lock and Dam

Figure 7. Spalled sections removed, 13 October 1983, Lower Monumental Lock and Dam

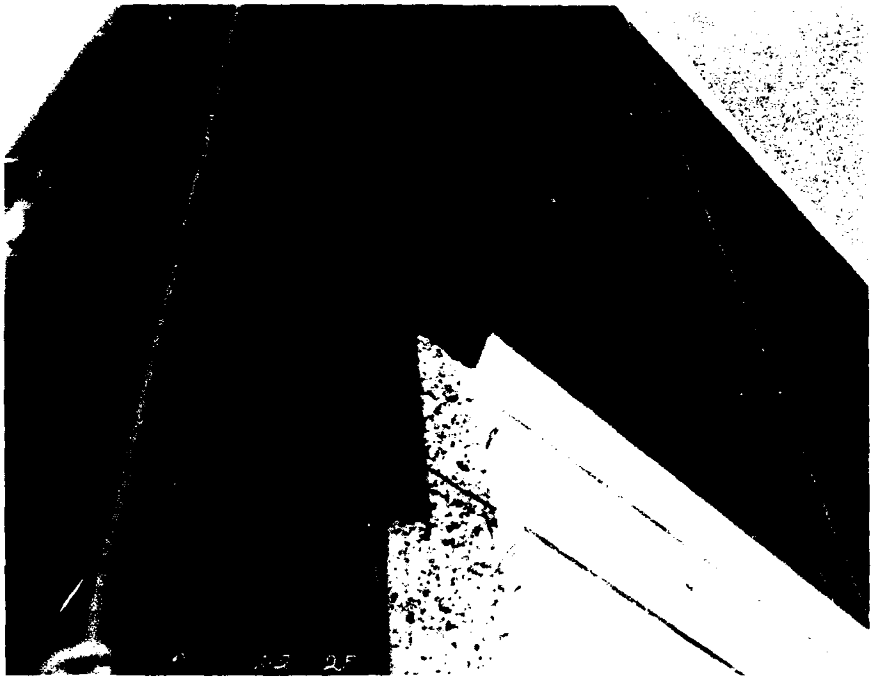


Figure 8. Surface prepared for repair, 17 November 1983, Lower Monumental Lock and Dam

Figure 9. Repair after form removal, 14 May 1984, Lower Monumental Lock and Dam

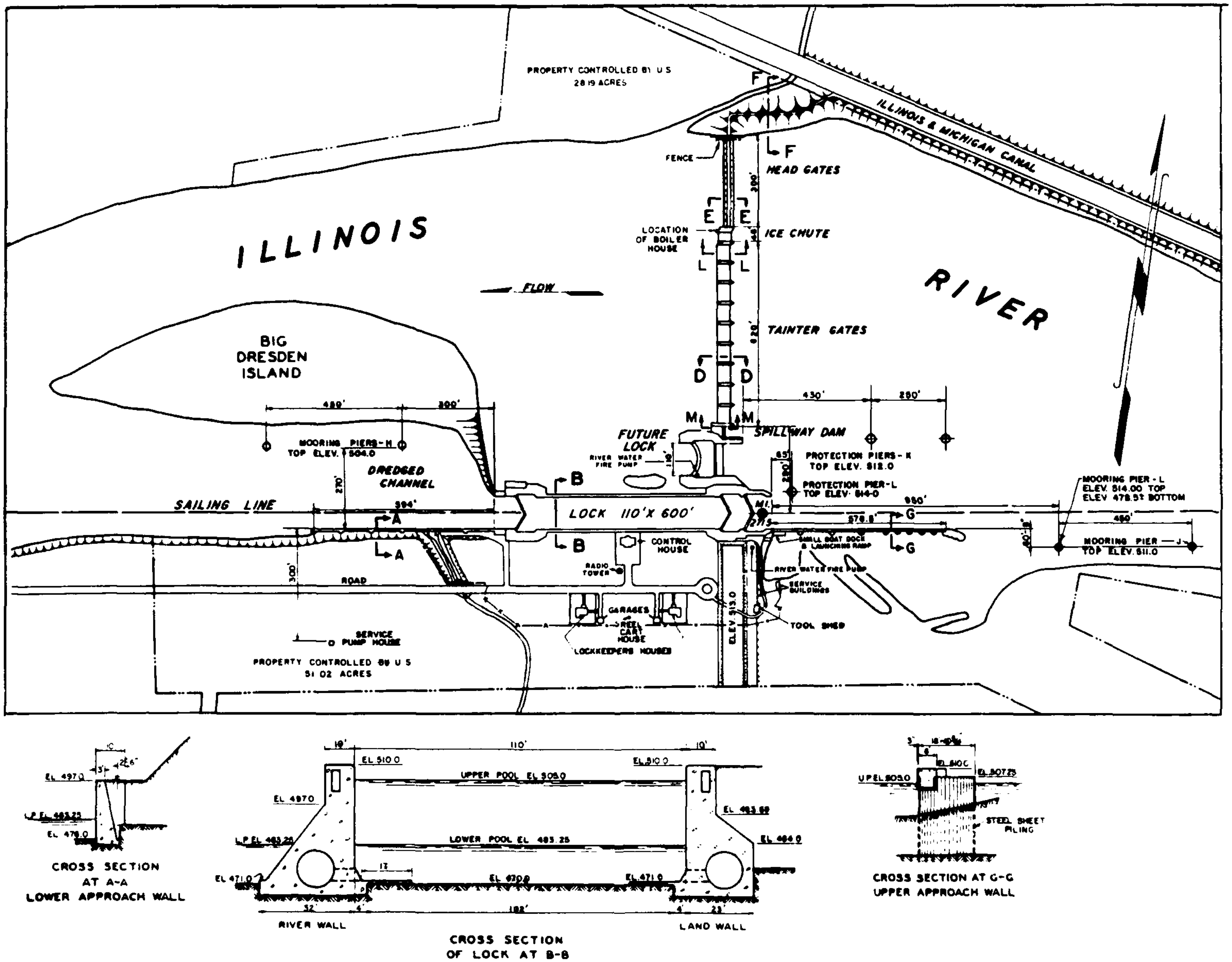


Figure 10. General plan, Dresden Island Lock and Dam

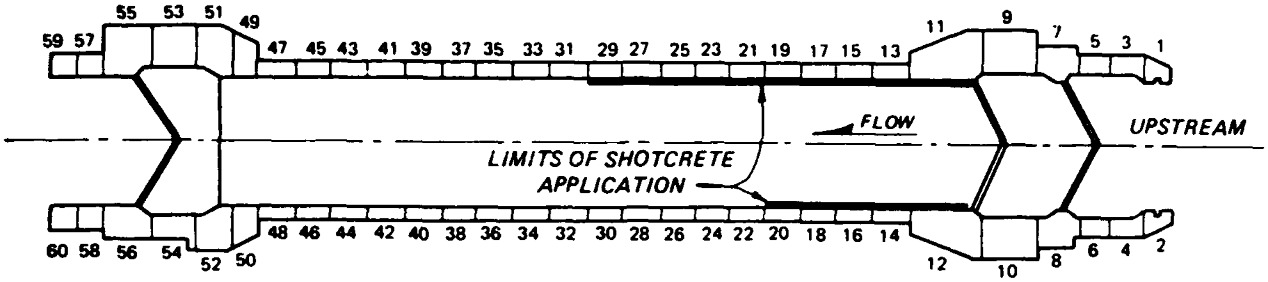


Figure 11. Limits of 1954 repair, Dresden Island Lock and Dam

the monolith joints where some spalling had occurred. The original structure did not contain expansion joints, but expansion joint material was incorporated in the relatively thin resurfaced section during 1954 rehabilitation. Because this material could not function as a joint but could absorb water, it probably contributed to the damage from cycles of freezing and thawing. The lock wall monoliths that had not been resurfaced were deteriorated an average depth of 7 in. Severe erosion was also noted at the monolith joints in the upper and lower guide walls.

16. A major rehabilitation plan was developed in 1977 (USAED, Chicago, 1977). Part of the rehabilitation was accomplished in 1978 under a contract awarded to J. M. Foster, Inc., of Gary, IN, for $4,444,444. Major features of the contract included: resurfacing the lock chamber walls, upper and lower gate bays and forebays, backside and top of river walls, and the upper and lower ends of the lock; repairing the upper and lower guide walls, upper and lower service gates, invert of lock culverts and miter gate machinery; and stabilizing the lower guide wall. Most of the work was accomplished during the scheduled shutdown during August and September 1978.

17. The downstream lock walls were completely refaced (Figure 12). This repair is described in detail in Technical Report REMR-CS-13 (McDonald

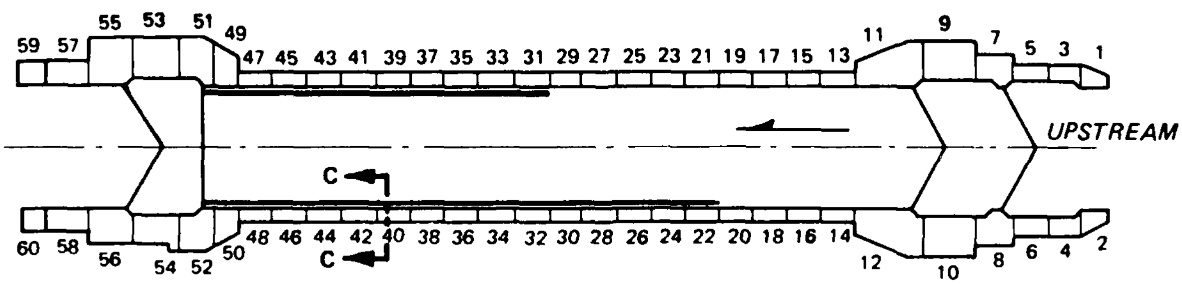


Figure 12. Limits of 1978 repair, Dresden Island Lock and Dam

1987). Because existing monolith joints were tight, only 30-lb asphalt-saturated felt paper was used in the monolith joints to serve as a bond

breaker. Each monolith joint had a 2- by 2-in. chamfer on each side. The full face of the joint was covered with curing compound before the joint material was placed (Figure 13).

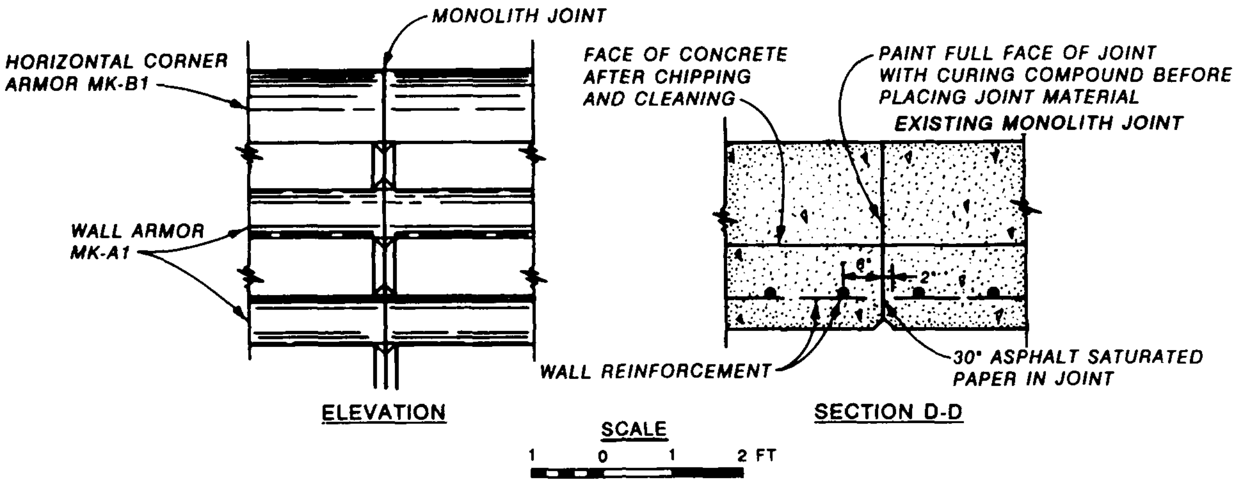


Figure 13. Monolith joint details for the 1978 repair, Dresden Island Lock and Dam

18. Approximately 25 percent of the upper and lower guide wall monolith joints had spalling severe enough to warrant repair. A saw cut, a minimum of 2 in. deep, was made 3 ft on each side of the joint. A minimum of 4 in., but typically 18 in., of concrete was removed by a hoe ram located on a barge (Figures 14 and 15). A cofferdam work box was erected and the water level inside was lowered (Figure 16). Anchor bars were then installed with an epoxy grout. Holes 1-1/2 ft deep and 1-1/8 in. in diameter were drilled and cleaned. A polyester resin cartridge was inserted and then the components were mixed with No. 6 anchor bars at a speed of 120 to 160 revolutions per minute. Three anchors were tested initially, and then 2 percent were tested thereafter. The specified anchor-pullout load was 8 tons or 90 percent of the yield strength of the grade 40 reinforcing steel. Two anchors on 2-ft vertical centers were installed on each side of the joint, one 3 in. from the joint and the other 3 in. from the saw cut (Figure 17). After the surface had been prepared, formwork was erected, and the conventional concrete mixture was placed to the full height with a tremie. (Concrete trucks were used to transport the concrete to the site.)

19. Although some of the joints in the lock chamber monoliths that were resurfaced in 1954 have spalled slightly, they are still serviceable. The remaining joints are in excellent condition.

Figure 14. Removing concrete from monolith joints on the lower guide wall with a hoe ram, Dresden Island Lock and Dam

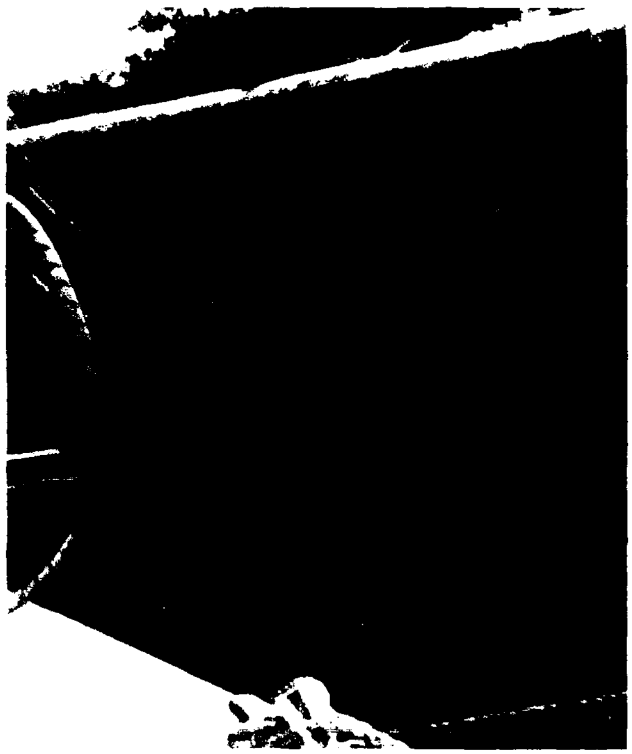


Figure 15. Closeup view of the hoe ram on the lower guide wall, Dresden Island Lock and Dam

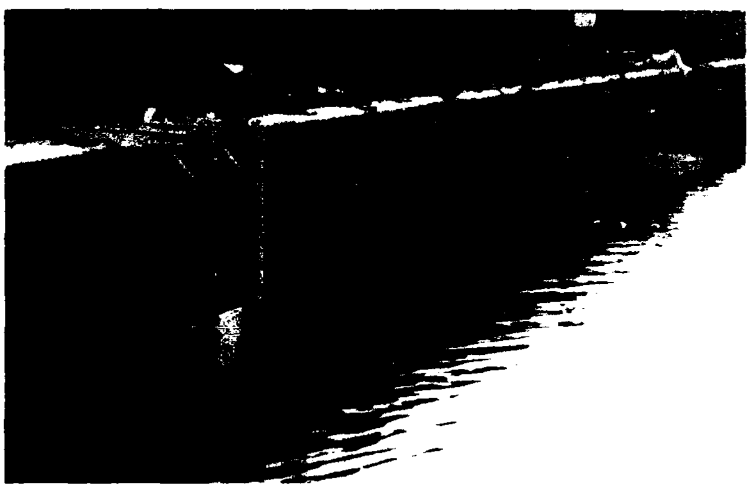
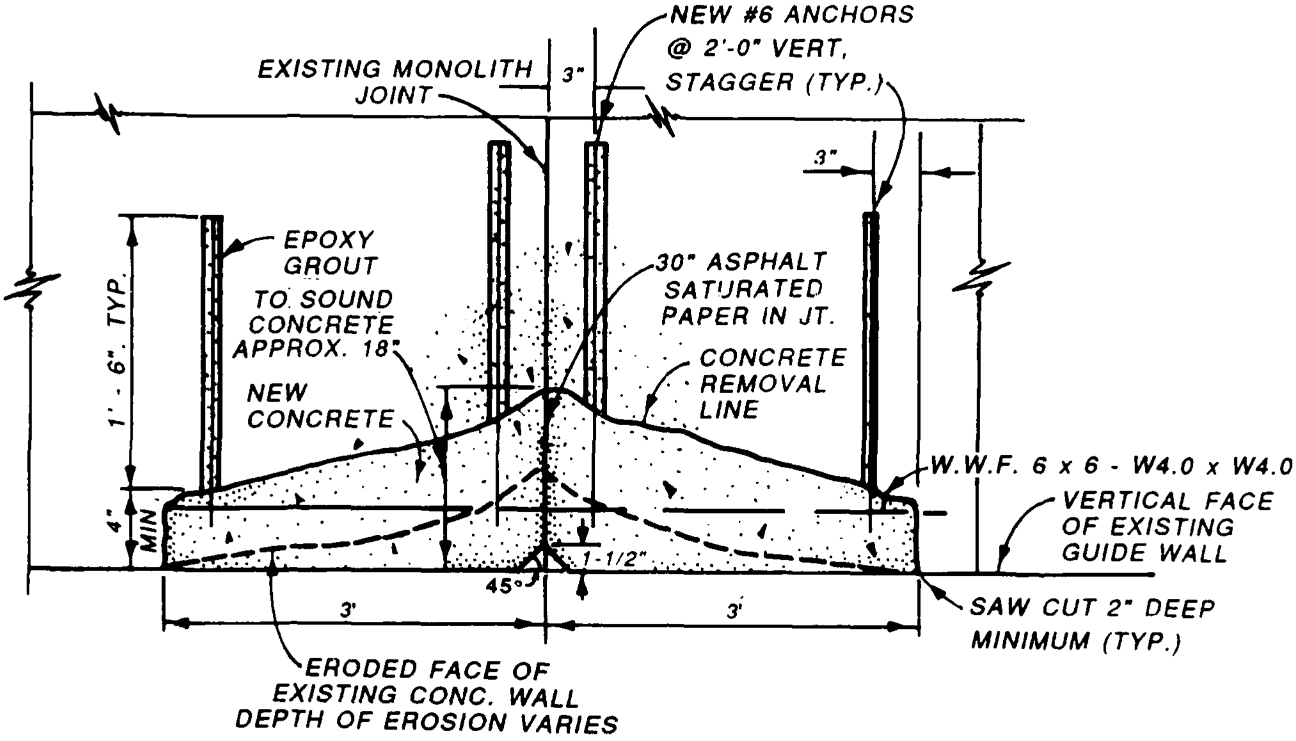


Figure 16. Cofferdam work box for joint repair on the lower guide wall, Dresden Island Lock and Dam



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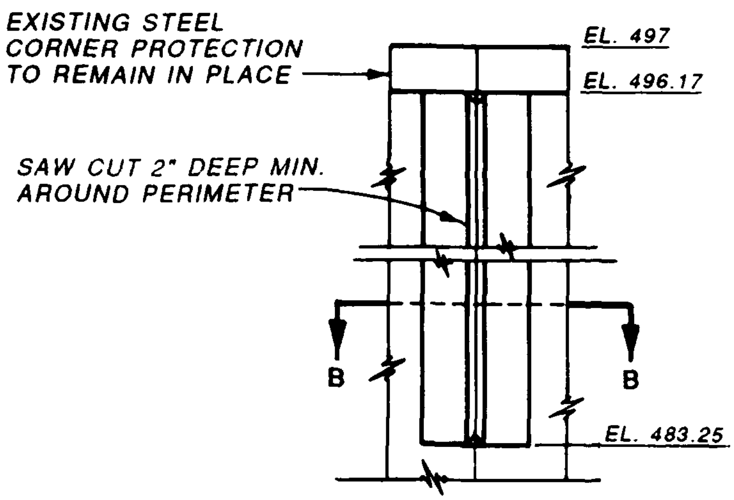


VERTICAL MONOLITH JOINT (TYP.)

Figure 17. Monolith joint repair details for the lower guide wall, Dresden Island Lock and Dam

Martis Creek Lake Dam

20. Martis Creek Lake Dam, located in northeastern California, was completed in 1972. The structure consists of an earthfill dam 113 ft high and 2,673 ft long at the crest and an uncontrolled concrete spillway capable of a maximum discharge of 4,060 cfs. Maximum storage capacity of the reservoir is 34,600 acre-ft.

21. In 1979 repairs were made to several monolith joints in the spillway where spalling had occurred. The spalls, caused by cycles of freezing and thawing (Figure 18), were repaired with epoxy mortar and Dri-Paк-It, a material similar to shotcrete.

Figure 18. Spalling at a monolith joint, Martis Creek Lake Dam

22. The epoxy mortar repairs were made by Adhesive Engineering Company. These repairs consisted of saw cutting the area around the spalls and removing the damaged concrete (Figure 19). The surface was then prepared to ensure a clean, dry surface, free of all loose material (Figure 20). The specifications required sandblasting or a high-pressure water jet. The repair area was primed with a thin coat of epoxy prior to the placement of the epoxy mortar. The epoxy, Concresive 1315, consisted of two components which were thoroughly mixed for 3 to 5 min with a mechanical mixer. Care had to be taken to ensure that no more epoxy was mixed than could be used before the pot life expired.

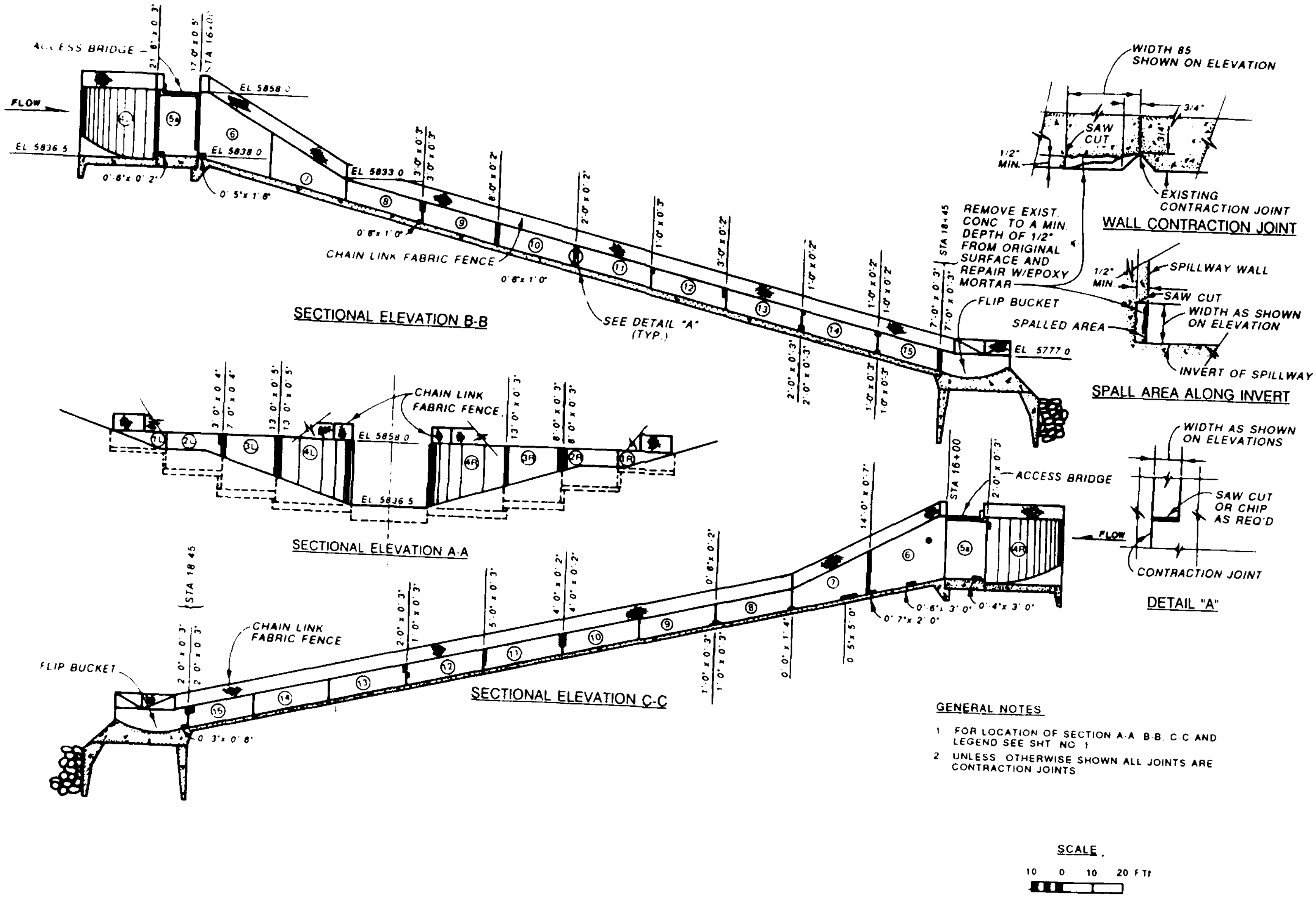


Figure 19. Joint repair details, Martis Creek Lake Dam

Figure 20. Prepared surface with saw-cut edges, Martis Creek Lake

For the epoxy mortar, aggregate was added after the two components were mixed. After the repair area was filled with epoxy mortar, the surface was finished with a trowel, and a curing compound was applied. Some thermal compatibility problems with the original material were encountered (Figure 21). After 10 years, approximately 50 percent of the repair is still in place.

23. The Dri-Pak-It repairs were made by the owner and distributor of Dri-Pak-It materials and equipment. First, the damaged concrete was removed with a chipping hammer without saw cutting the edges. All loose material was removed (Figure 22). Next, a mixture of water and Ali/Cite, a waterproofing compound, was applied to the area to be repaired just before the Dri-Pak-It material was applied (Figure 23). The Dri-Pak-It mixture had a 3-percent moisture content and a ratio of 1 part cement to 1-1/2 parts sand. The sand and cement material was then applied while simultaneously being sprayed with the water and Ali/Cite mixture (Figure 24). The pneumatic application system consisted of a hopper-fed gun with one nozzle to dispense the sand and cement mixture and another to dispense the mixing water. The mixing water was held

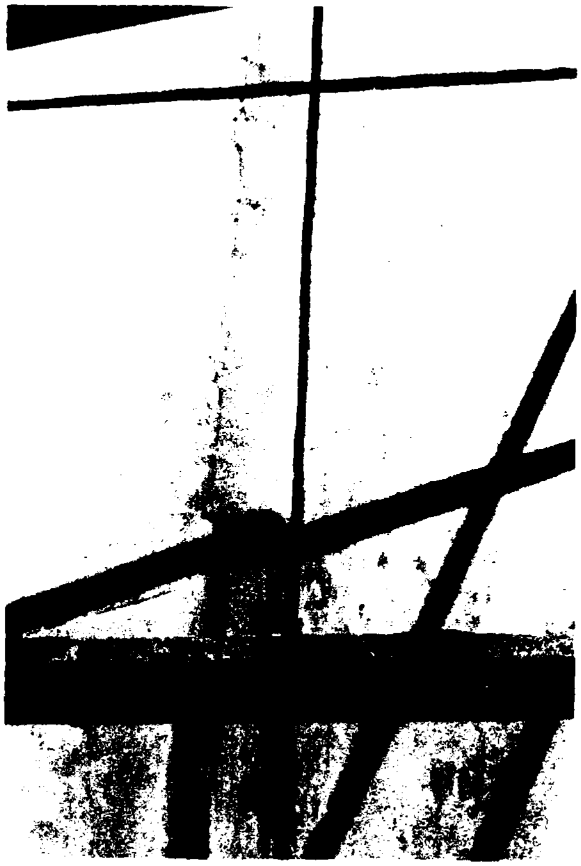


Figure 21. Epoxy mortar repair, Martis Creek Lake Dam

Figure 22. Surface preparation, Martis Creek Lake Dam

Figure 23. Water and Ali/Cite mixture being applied, Martis Creek Lake Dam

Figure 24. Dry-Pak-It repair material being applied, Martis Creek Lake Dam

in a tank which contained 6 gal of water mixed with 1 qt of Ali/Cite. The required air pressure ranged from 75 to 80 psi. A steel trowel was used to remove excess material. The surface was consolidated with a sponge rubber float saturated with Ali/Cite and water followed by a final pass with a steel trowel. Finally, Clear Seal Concentrate was brushed on to cure the repair. This repair has held up fairly well over the past 10 years although some loss of bond has occurred (Figure 25). District personnel were very impressed with the simplicity of this system.

24. In 1984 another spall repair (although not at a monolith joint) was made with the Dri-Pak-It system. This system was essentially the same as the one used in 1979 with the exception of the addition of a vibrating hopper, a new surface preparation product, and a new mix-water admixture. Because the temperature during placement was 40° F, twice as much mix-water admixture was used to increase the rate of set time and thereby preclude the need for insulating the repair overnight. These repairs are also holding up fairly well.

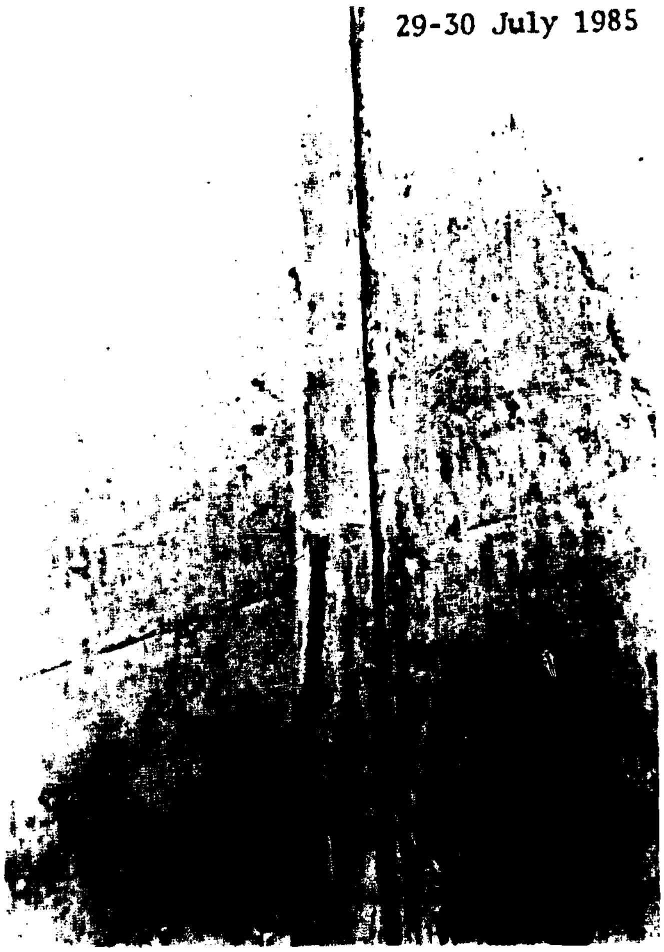


Figure 25. Loss of bond on Dry-Pak-It repairs, Martis Creek Lake Dam

25. In 1985 repairs were made between monolith joints 7/8, 8/9, 10/11, and 12/13. Project personnel used Thorobond, a liquid bonding agent, to make the repairs, which are not holding up well and need to be redone (Figure 26). This failure has been attributed to improper surface preparation and questionable material selection.

26. Periodic Inspection Report No. 9 (USAED, Sacramento, 1986), recommended that a 1-in.-deep saw cut be used on all future spall repairs to prevent feathered edges. The inspection report also recommended that if reinforcing bars are exposed when deteriorated concrete is removed, the entire circumference of the reinforcing bar should be exposed so that the repair material will achieve a better bond.

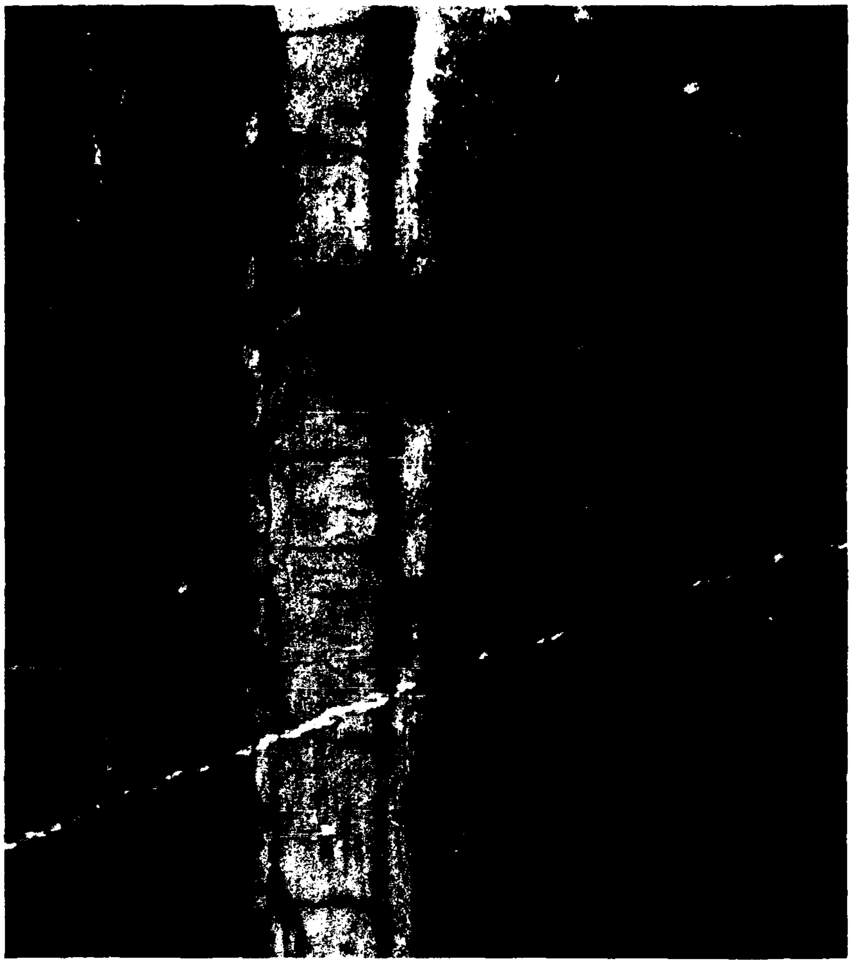
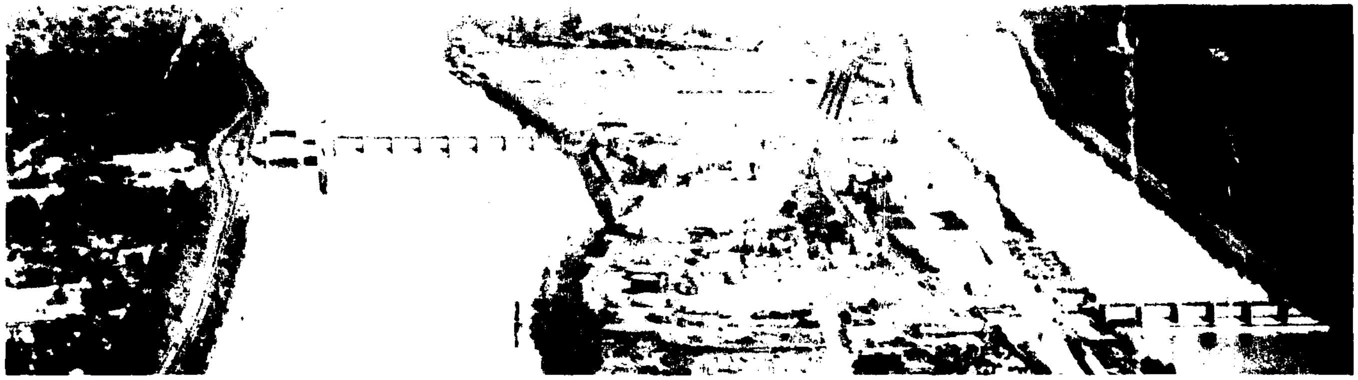


Figure 26. Deteriorated Thorobond repairs, Martis Creek Lake Dam

Emsworth Locks and Dams

27. Emsworth Locks and Dams are located in southwestern Pennsylvania on the Ohio River below the confluence of the Allegheny and Monongahela Rivers at Pittsburgh. They consist of two structures, one on each side of Neville Island (Figure 27). The main channel structure is located 6.2 miles below the head of the river and consists of a dam and two parallel locks (Figure 28). The dam is 967.42 ft long with a fixed weir and 709.00 ft of controlled spillway consisting of eight gated 100-ft-long sections. The riverside lock is 360 ft long and 56 ft wide. The landside lock is 600 ft long and 110 ft wide. Both locks have an 18-ft lift. The back channel structure, which is 6.8 miles downstream of the head of the Ohio River, consists of a controlled 750-ft-long spillway with six gated sections each 100 ft in length (Figure 29).



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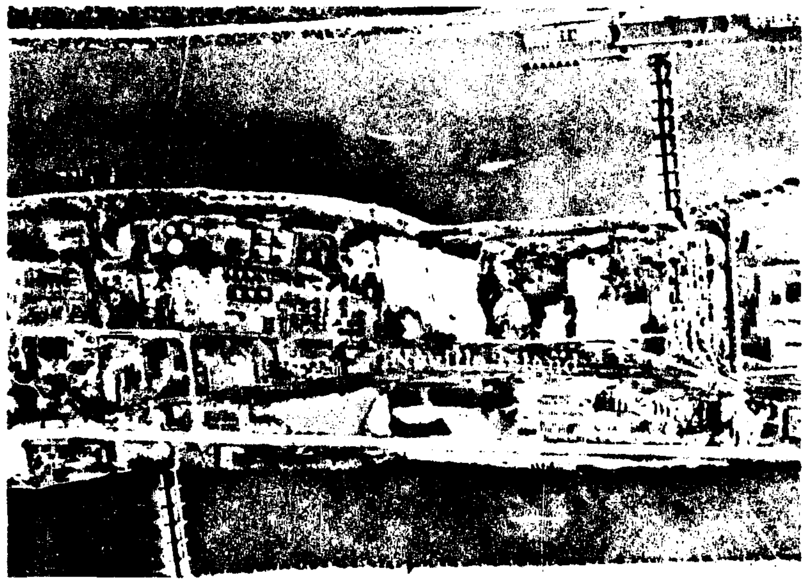

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Figure 27. Aerial view of Emsworth Locks and Dam

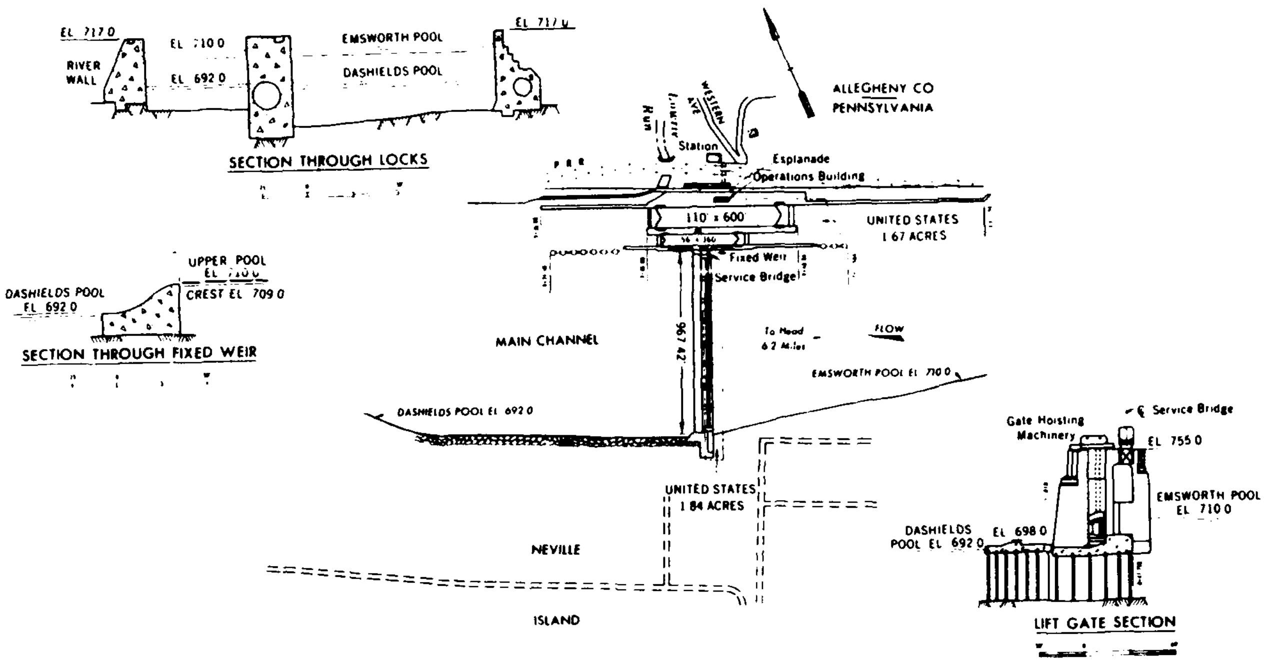


Figure 28. Main channel structure, Emsworth Locks and Dams

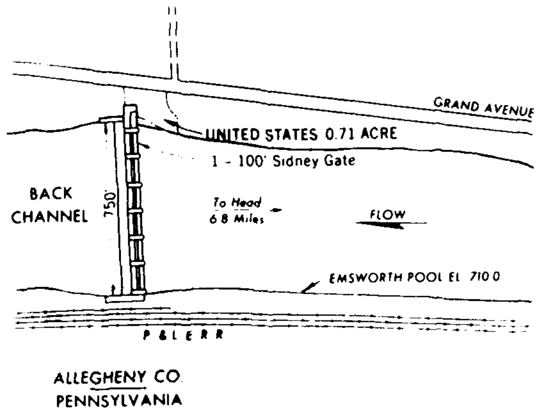


Figure 29. Back channel structure,
Emsworth Locks and Dams

Construction took place between 1919 and 1922. The dams were reconstructed between 1935 and 1938 to provide gated crests and to raise the Emsworth Pool by 7 ft. The lock chamber walls were resurfaced with shotcrete in 1931 and 1956 (USAED, Pittsburgh, 1971).

28. The first periodic inspection (USAED, Pittsburgh, 1971) indicated that the lock wall concrete was in poor condition, especially at the monolith joints (Figure 30). The shotcrete in some of the joint locations had completely failed because of barge impacts or damage from cycles of freezing and thawing. The inspection team recommended that routine maintenance be performed as it became necessary even if it had to be performed ahead of schedule.

29. Because of the poor condition of the project, the District conducted a study in 1973 to determine the quality of the concrete. The study, which included core drilling and testing, petrographic examination, borehole photography, pulse velocity tests, and a crack survey, indicated that the concrete below the deteriorated surface was of moderate quality with an average compressive strength of 4,000 psi (Denson and Buck 1974).

30. An engineering condition survey and structural investigation were conducted from 1974 to 1976 (Pace 1976) to determine the need for replacement or rehabilitation of the structure. The survey found that the lock walls did not meet contemporary stability requirements and that the deteriorated concrete surface would result in deterioration of the sound interior concrete if nothing was done.

31. A demonstration repair using Fibercrete, a steel fiber-reinforced shotcrete, was conducted in November 1980 on a section of the original upstream guide wall. Concrete was removed from one monolith joint until a vee-shaped 6-in.-deep joint was formed (Figure 31). The joint and surrounding surface were then prepared with a high-pressure water jet to achieve a clean and wet surface. A strip of joint filler was placed in the vee joint. The dry-mix shotcrete, consisting of sand, cement, and 1-in.-long steel fibers, was fed into the apparatus at a rate of 120 to 180 lb/min. Water was introduced at the nozzle at 70 to 80 psi. After the joint was completely filled, a 1- to 2-in. overlay was applied over the entire section (Figure 32). Rebound was nominal and did not pose a problem during application. No curing was required unless the temperature exceeded 90° F, at which point wet burlap would be used for 24 hr. Manufacturers claimed that the use of steel fibers

Figure 30. Examples of typical joint deterioration, land lock chamber wall, 1981, Emsworth Locks

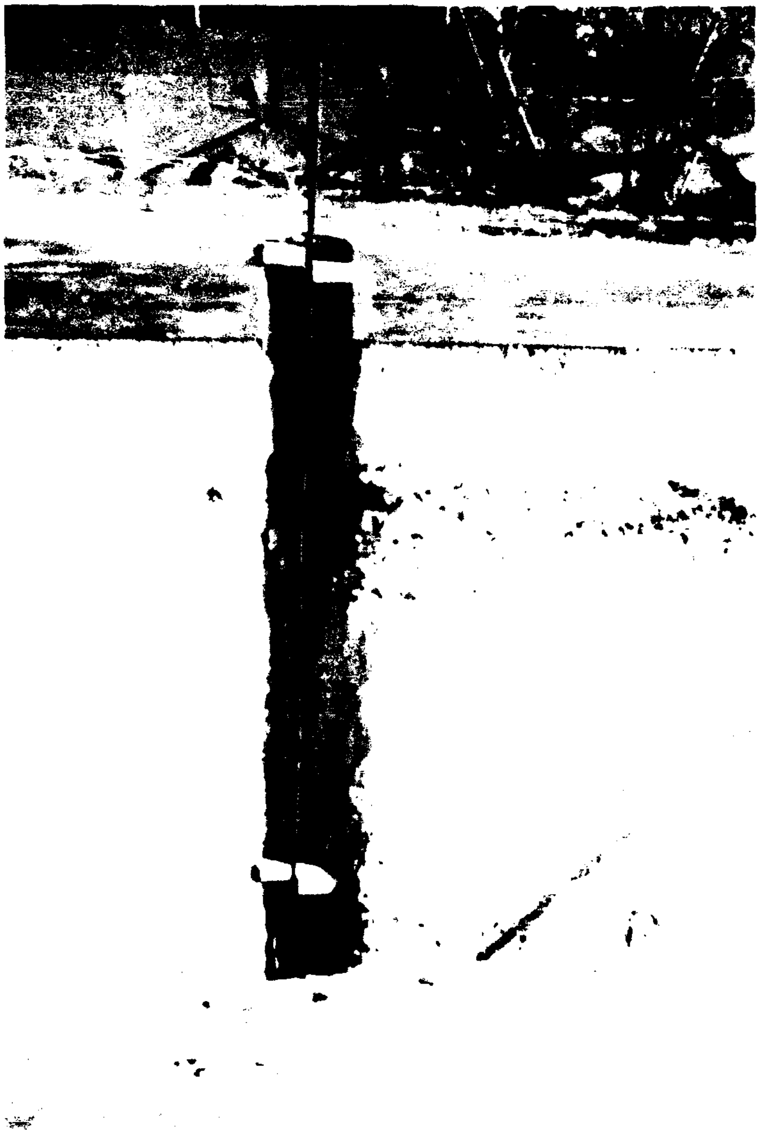
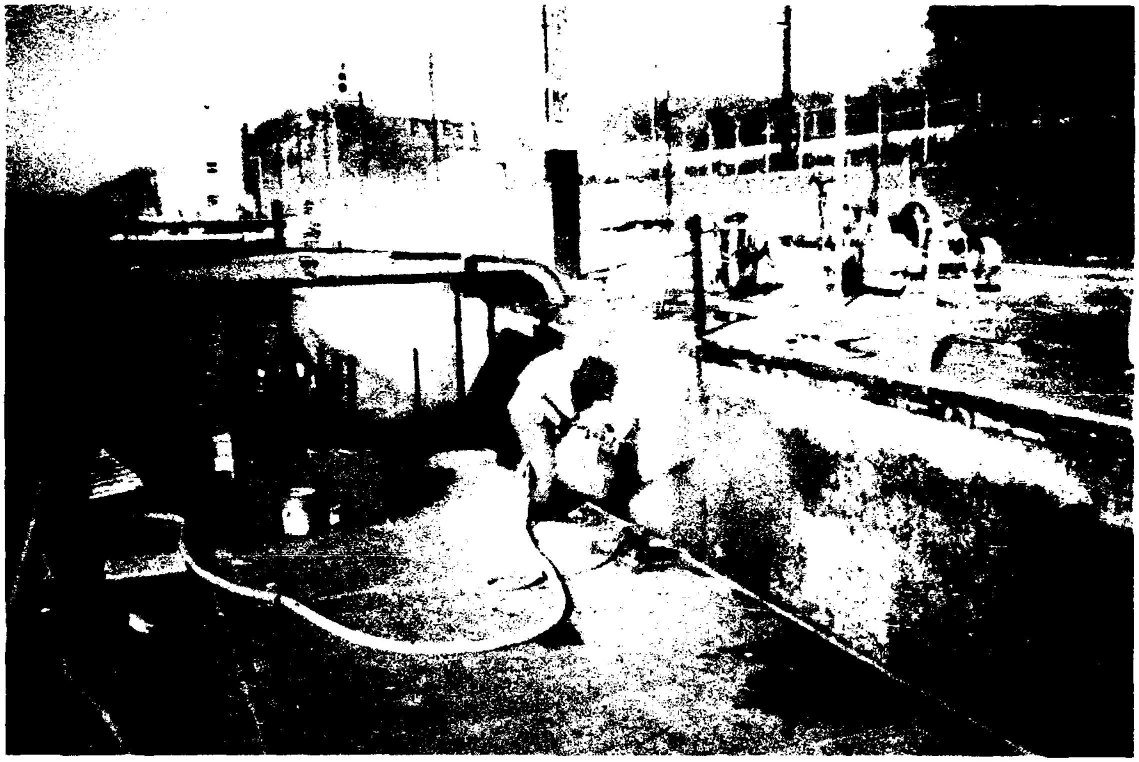


Figure 31. A portion of the prepared test section prior to Fibercrete application, November 1980, Emsworth Locks



a. Application

b. Portion of completed test section

Figure 32. Fibercrete test section, November 1980, Emsworth Locks

improved the compressive strength by 30 percent, flexural strength by 200 percent, and tensile strength by 200 percent as compared to conventional shotcrete.

32. After 3 months the test section exhibited impact failures, abrasion erosion, and delamination. One explanation for the poor performance was that the prepackaged mixture used for the demonstration contained only 60 lb/cu yd of fibers. For the actual repair 200 lb/cu yd of fibers would have been specified.

33. A feature design memorandum for a major rehabilitation of the Emsworth Locks and Dams with the intent of extending the service life of the structures for another 25 years was issued in 1980 (USAED, Pittsburgh, 1980). The contract for the rehabilitation was awarded to the low bidder, Morris-Knudsen, for $24,285,989 in October 1981. Responsibility for administering the contract was transferred to the US Army Engineer District, Huntington, in November 1981.

34. Navigation traffic through the locks was maintained during the rehabilitation: work on the river chamber was completed before work on the land chamber was begun. Because the land chamber was the primary chamber by virtue of its size, work which would have interrupted navigation traffic was limited to two 30-day periods. The lock walls were refaced with conventional concrete, resurfaced with shotcrete, or coated with shotcrete depending upon the degree of damage in a particular area (Figure 33). The repair extended from 1 ft below the lower pool elevation to the top of the wall.

35. The conventional concrete-refacing repairs began with the use of explosives to remove the deteriorated concrete to a depth of 1 ft. This repair technique is described in detail in Technical Report REMR-CS-13 (McDonald 1987) and summarized here. The reinforced concrete was designed to have a compressive strength of 3,000 psi at 28 days, a maximum water-cement ratio of 0.50, and a 1-1/2-in. maximum size coarse aggregate. The only special treatment given to the monolith joints during the concrete repairs was the addition of a bituminous expansion joint material and a 45-deg chamfer on each side of the 1-1/8-in.-deep joint (Figure 34).

36. The areas to be resurfaced with shotcrete were first prepared by removing the existing material to a 3-in. minimum depth. The shotcrete mixture was designed to have a compressive strength of 4,000 psi at 28 days, a total air content between 4 and 7 percent, and a 2- to 4-in. slump. An

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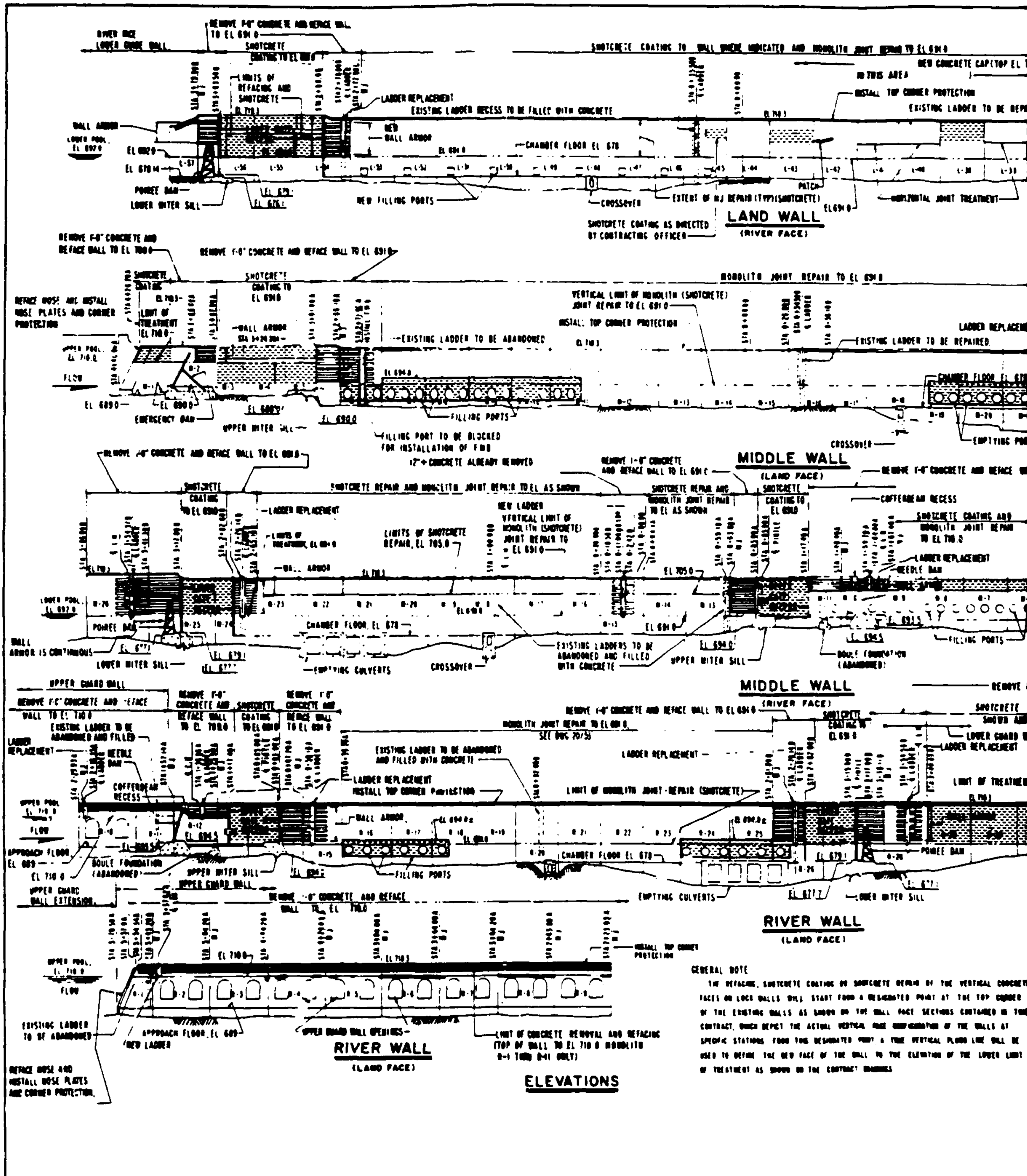
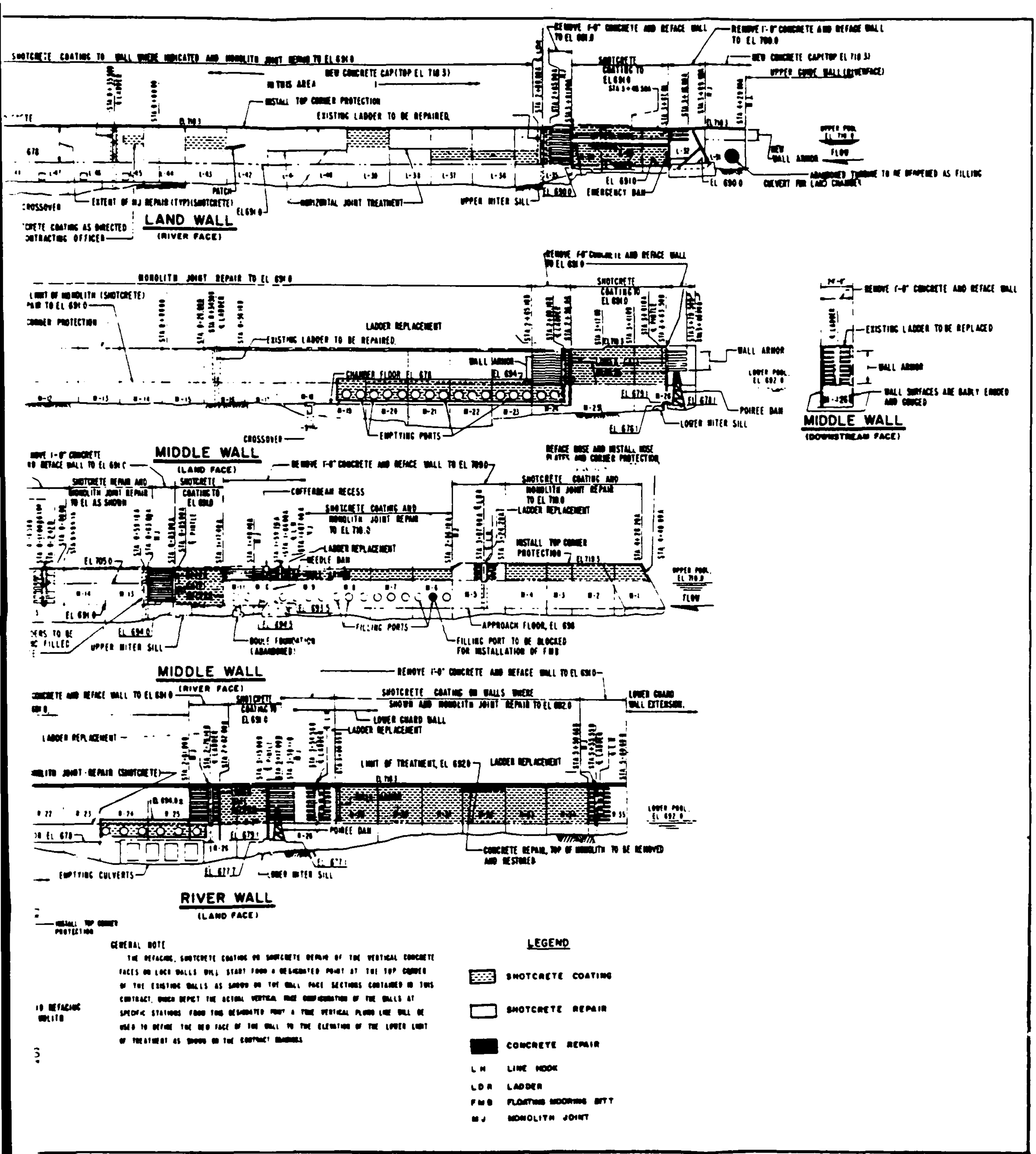


Figure 33. Elevations, lock chamber wall refacing,



.ations, lock chamber wall refacing, Emsworth Locks

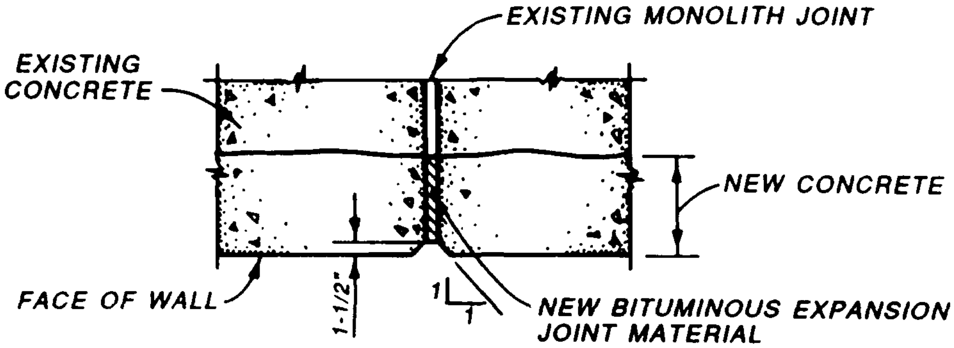


Figure 34. Conventional concrete-refacing monolith joint repair details, Emsworth Locks

accelerator was used to minimize sagging. Hooked dowels were installed on both sides of the monolith joint. A bituminous expansion joint material was placed at the joint and the corners were beveled (Figure 35).

37. Some areas received a shotcrete 3/4-in.-thick coating, while other areas had only the monolith joints repaired. At these monolith joints, 6 in. was the minimum removal depth (Figure 36). Hooked dowels were grouted on both sides of the joint with polyester resin. Expansion joint material was installed and the joint was filled leaving a 45-deg bevel, 1-1/8-in. deep on both sides of the joint (Figure 37).

38. A condition survey of Emsworth Locks and Dams (Stowe and Poole 1986) indicated that all of the monolith joints repaired during the 1982-1983 rehabilitation were in good condition (Figure 38).

Lock and Dam No. 2, Mississippi River

39. Lock and Dam No. 2 is located on the Mississippi River 1.4 miles upstream from Hastings, MN. The structure consists of a concrete dam, two locks, and an earth dike. The dam includes a controlled section consisting of twenty 30-ft-wide tainter gates and 100 ft of uncontrolled spillway. The landward lock is 110 ft wide and 600 ft long with a 12-ft lift. The riverward lock, which is 110 ft wide and 500 ft long, is no longer used (Figures 39 and 40). The original structure (dam, dike, and riverward lock) was constructed from December 1928 to June 1931. Construction on the landward lock began in 1941 but was not completed until 1948 because of a suspension of civil works construction during World War II. Once completed, the landward

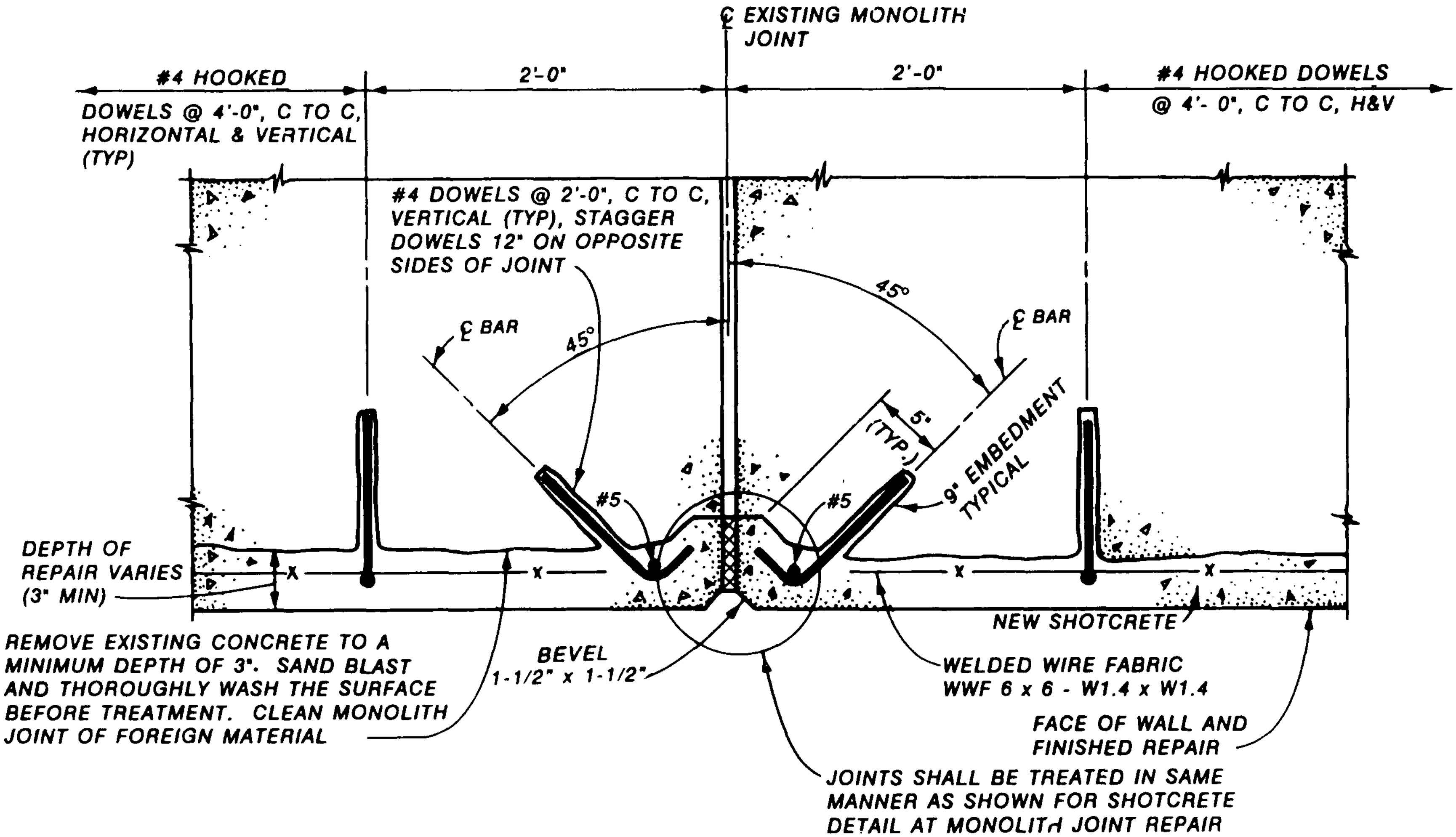


Figure 35. Shotcrete resurfacing monolith joint repair details, Emsworth Locks

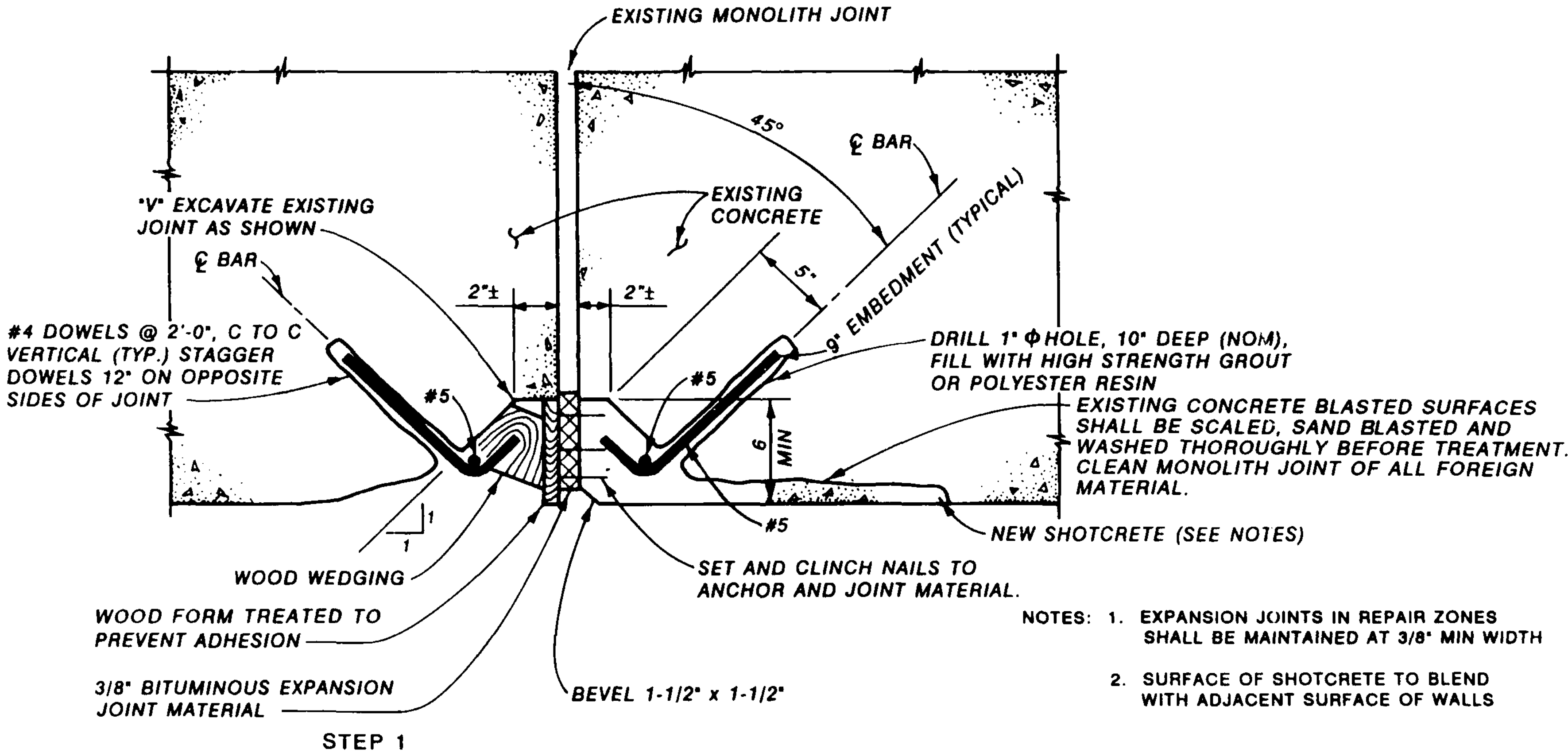
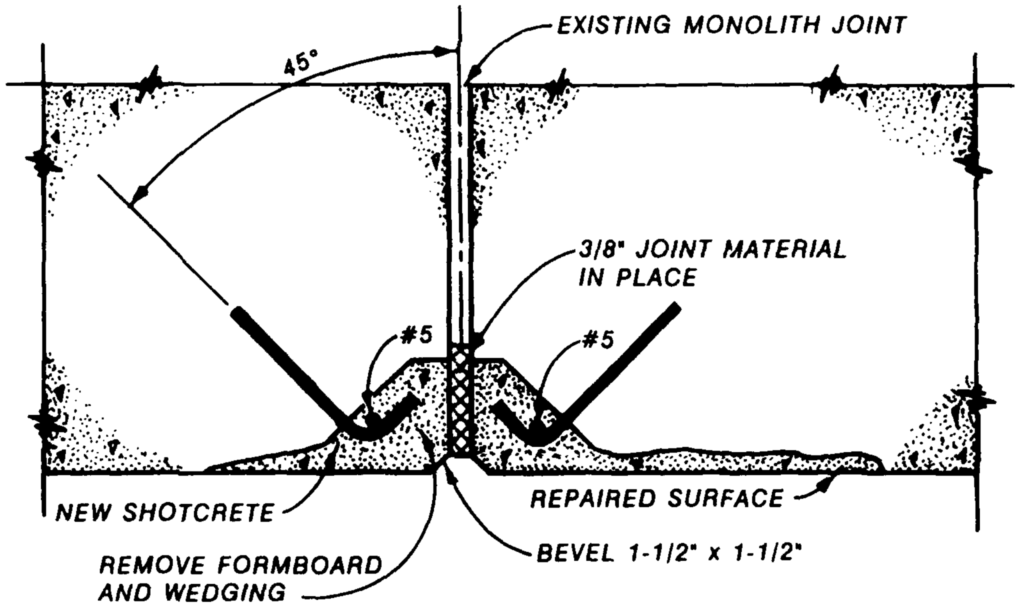


Figure 36. Shotcrete monolith joint repair details, Emsworth Locks



STEP II

Figure 37. Shotcrete monolith joint repair details, Emsworth Locks

Figure 38. Main channel structure, Emsworth Locks and Dams

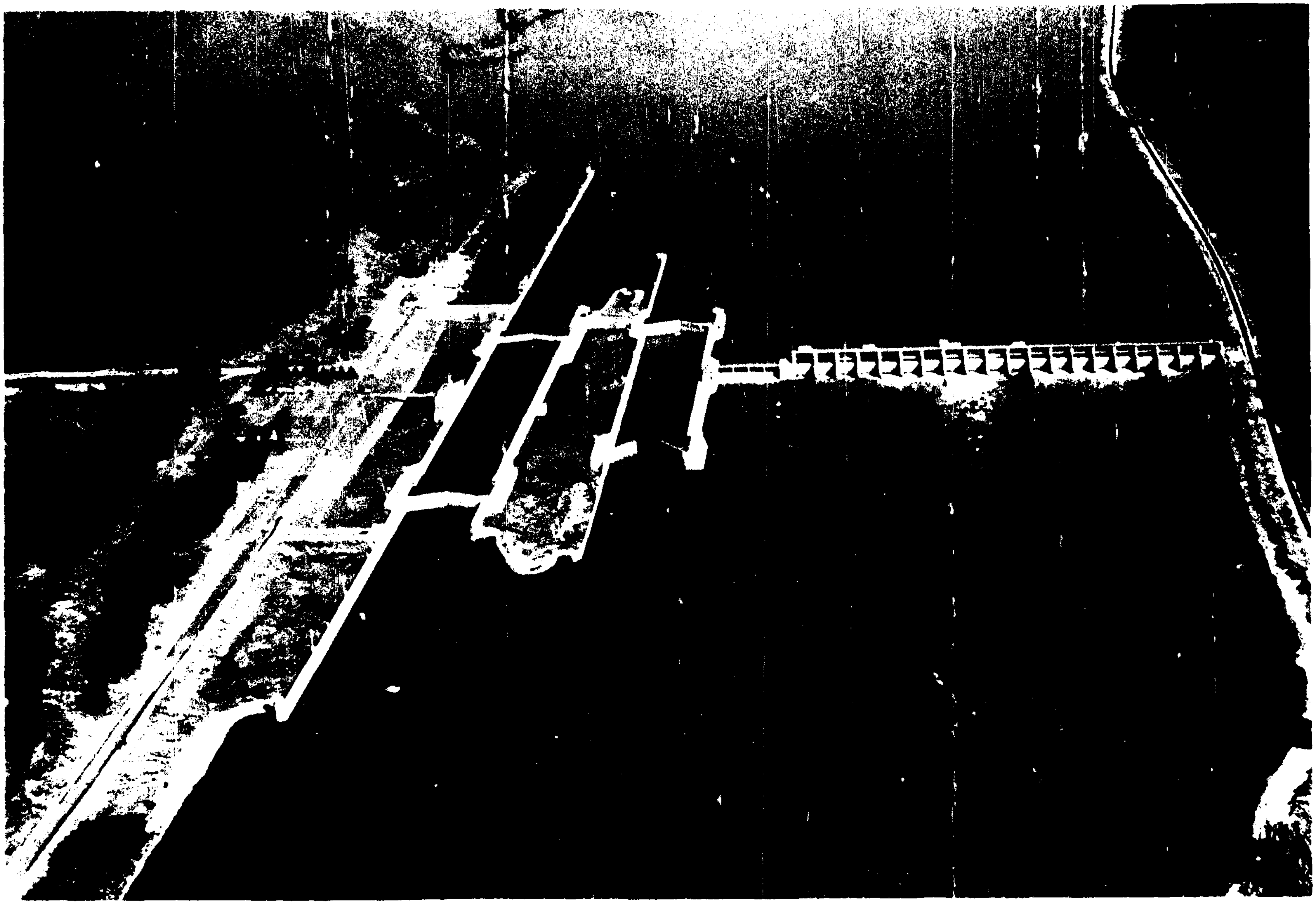


Figure 39. Aerial view, Lock and Dam No. 2

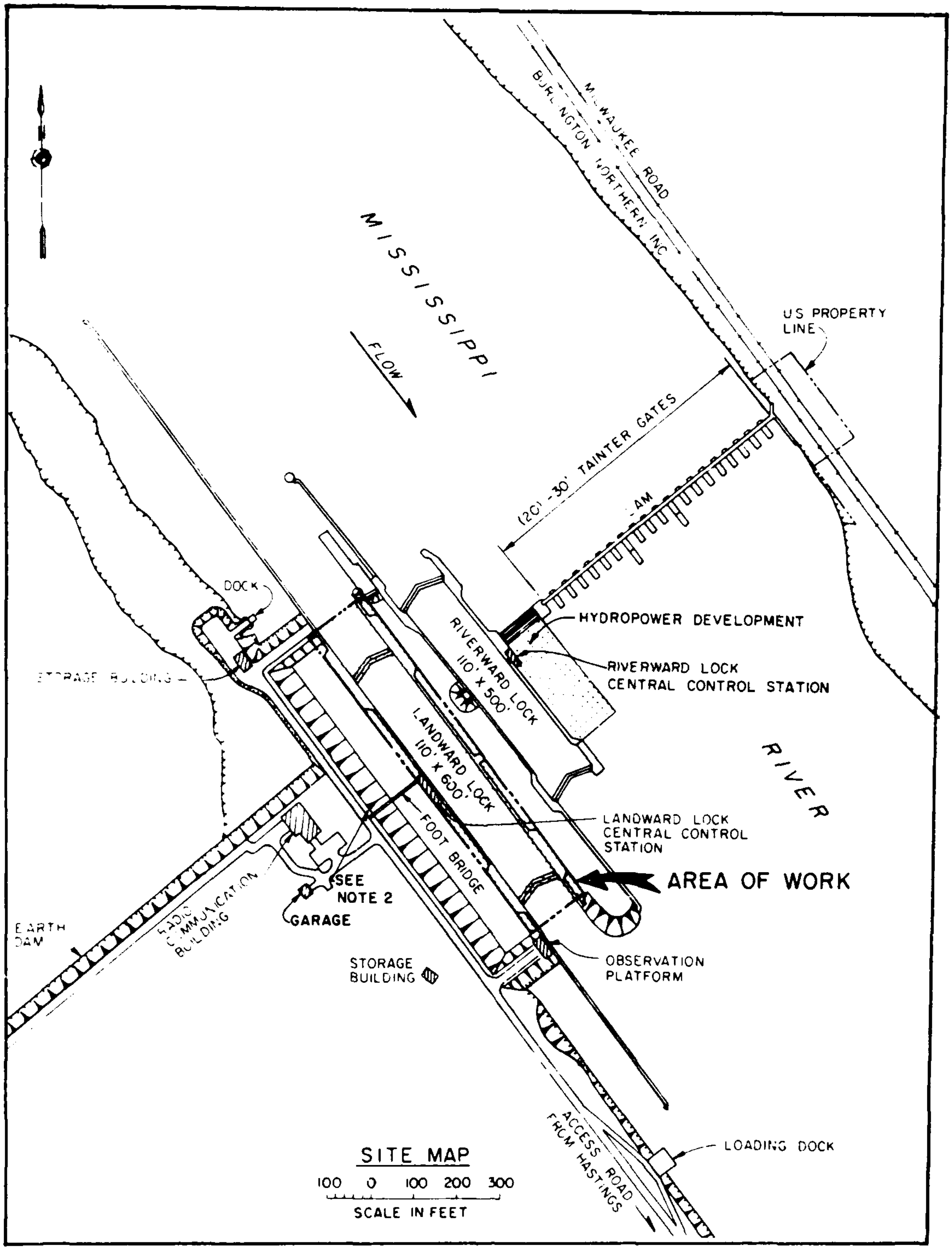


Figure 40. General plan, Lock and Dam No. 2

lock replaced the riverward lock, no longer in use because of its nonstandard size. Because of foundation problems with the riverward lock, some rotation of the lock walls caused the miter gates to operate out of tolerance. The concrete in the lock walls consisted of 423 lb/cu yd of cement, 2-in. maximum size aggregate, and a 0.55 water-cement ratio. Testing of several 6-in. cores indicated a compressive strength ranging from 5,300 to 8,080 psi. The concrete was nonair entrained.

40. During the first periodic inspection (USAED, St. Paul, 1971), spalling at all the vertical monolith joints was observed inside the chamber of the landward lock. This spalling was attributed to impacts from navigation traffic and cycles of freezing and thawing. A limited amount of spalling on the vertical joints in the unused riverward lock was also observed. A concrete condition report (USAED, St. Paul, 1985) indicates that the vertical monolith joints in the landward chamber had continued to deteriorate. The spalling was described as moderate to heavy and extending from the lower pool elevation to the top of the lock wall (Figure 41). The joints on the river wall were more deteriorated, possibly because they were exposed to the winter sun and, therefore, subject to more cycles of freezing and thawing than the joints on the land wall. The report recommended that the spalling at the lock wall monolith joints be repaired to prevent further deterioration.

41. In 1984 a major rehabilitation effort began. The intent of this effort was to extend the service life of the structure for another 50 years. Under contract, Harza Engineering Company prepared a design analysis report (USAED, St. Paul, undated) for the concrete and wall armor repairs. While some of the spalling around the monolith joints could be attributed to impacts from navigation traffic, most of the deterioration was attributed to cycles of freezing and thawing in which the concrete had become saturated from water in the sponge rubber joint filler. Four repair alternatives were evaluated for the following characteristics: cost, resistance to cycles of freezing and thawing, and resistance to damage from impact. Alternative 1 proposed removing the damaged concrete and patching with a polymer-cementitious mortar. In alternative 2, a minimum 4-in. saw cut would be made on each side of the joint and the deteriorated concrete removed. Bolt anchors on 6-in. centers would be installed and a glass fiber-reinforced polymer mortar would be placed. The joint would be recessed 1/2 in. Alternative 3 consisted of a saw cut 8 to 10 in. on each side of the joint. Anchors and wall armor would be installed

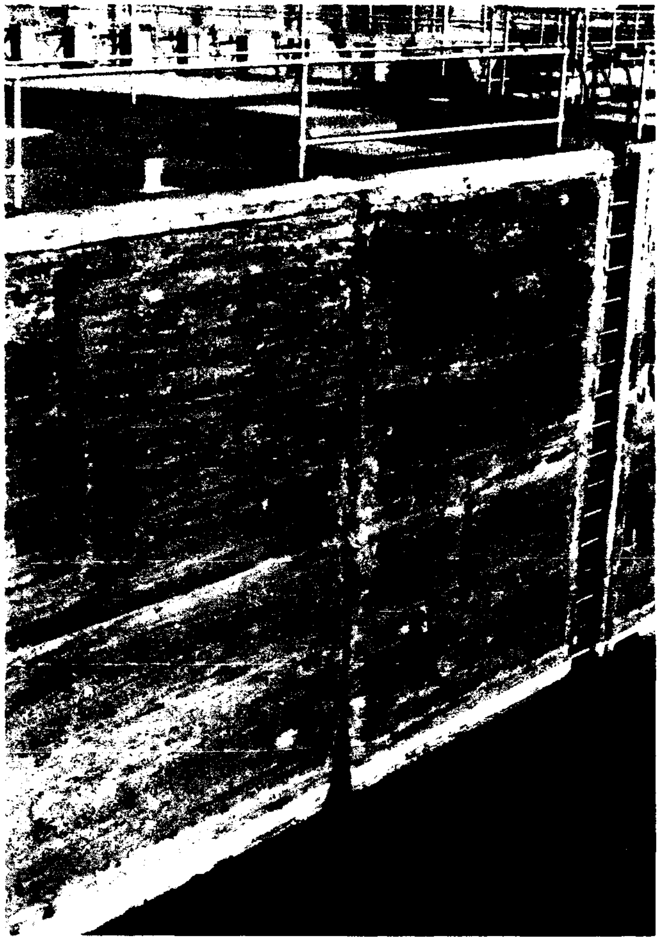


Figure 41. Typical monolith joint deterioration, Lock and Dam No. 2

and a nonshrink grout would be placed. Alternative 4 was similar to alternative 2 except for the addition of horizontal wall armor installed 5 ft on each side of the joint. Alternative 2 was selected as the optimum repair solution. Fiber-reinforced acrylic polymer modified concrete (FRAPMC) was recommended as the repair material because of its characteristics:

Compatibility with existing materials
Bonding properties
Strength
Vapor transmission
Durability
Low shrinkage potential
Reduced shrinkage cracks

42. The repairs to the locks were completed in two stages. Stage I consisted of repairs to the concrete and operating machinery, and Stage II

involved the installation of new operating machinery and controls. Repairs to the lock wall were completed in Stage I when the project was dewatered during the 1986-1987 winter. Work began with the identification and removal of the deteriorated concrete around the joints. (Identification was made by visual examination and the use of a sounding hammer.) A 1-1/2-in.-deep saw cut was made a minimum distance of 4 in. from the joint. The cut was angled from the joint to "lock" the repair into the existing concrete, except for the top joint which was cut perpendicular to the face of the wall. The deteriorated concrete was removed with hand-held jackhammers which allowed the mechanics to actually feel the deterioration beyond the minimum removal depth of 2 in. (Figures 42 and 43). The original joint was filled with 18 in. of asphalt-impregnated foam rubber secured with nails. Of a variety of methods, an electric chain saw was determined to be the best tool to remove joint material. Mechanical anchors 1/2 in. in diameter were installed on 9-in. centers on both sides of the joint. Mechanical anchors were selected because their full strength was realized immediately after they were set. After the anchors were set, all loose materials were blown or brushed from the surface to be patched. Because the temperatures were well below freezing, special provisions were made to protect the concreting operations (Figure 44). The cement and polymer were stored inside a heated enclosure; the aggregate was stored in a heated stockpile, and the scaffolding around each joint was enclosed and heated (Figure 45). The FRAPMC was placed in 7-ft lifts and consisted of a modified cement, liquid polymer, 3/8-in. granite aggregate, and 2-1/4-in. polypropylene fibers. The cement and polymer were in prepackaged units which made mixing very simple. Each unit had a volume of 1/2 cu ft to which 30 lb of aggregate and 1 oz of fibers were added. To get a self-leveling mixture that could easily be placed and consolidated, 25 to 30 percent more polymer had to be added. Because FRAPMC would adhere to a wooden form coated in form oil, the forms required a 6-mil polyethylene lining. Even with a form release agent specifically designed for use with acrylic modified concrete, the FRAPMC still bonded to the form if polyethylene was not used. After only 2 hr, the form was removed and used for the next lift. The joint was recessed from the face of the lock wall with a bevel 3-1/2 in. by 1/2 in. and a 45-deg chamfer at the joint edge (Figures 46 and 47). The 1/2-in. gap between monoliths was maintained during placement with the use of gatorboard (a styrofoam core with a thin plastic face on both sides). After the FRAPMC had cured sufficiently for

Figure 42. Removing deteriorated concrete, Lock and Dam No. 2

Figure 43. Joint with deteriorated concrete removed, Lock and Dam No. 2

the gatorboard to be removed, the soft styrofoam center was removed and the plastic faces, previously coated with form oil, released easily. The expansion joint was filled with a water-activated polyurethane foam grout and sealed with a polyurethane joint sealant (Figures 48 through 50).

43. These FRAPMC joint repairs continue to perform well. The same technique and material was used by the USAED, St. Paul, to repair similar spalling at Lock and Dam No. 3 during the 1987-1988 winter.

Algiers Lock

44. Algiers Lock is located in Orleans Parish, LA, on the west side of the Mississippi River about 7 miles southeast of New Orleans. The lock is a reinforced-concrete U-frame with sector-type gates in each gate bay. The lock has a usable chamber 75 ft wide and 755 ft long (Figure 51). Because of unfavorable foundation conditions, the lock was founded on wood pilings which

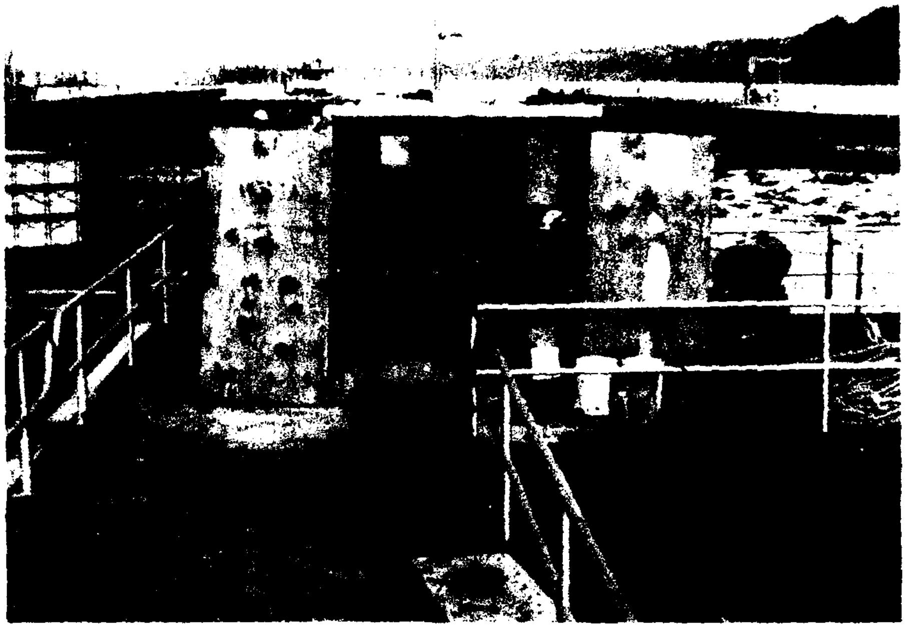


Figure 44. Heated concrete-mixing shelter, Lock and Dam No. 2

Figure 45. Enclosed and heated scaffolding, Lock and Dam No. 2

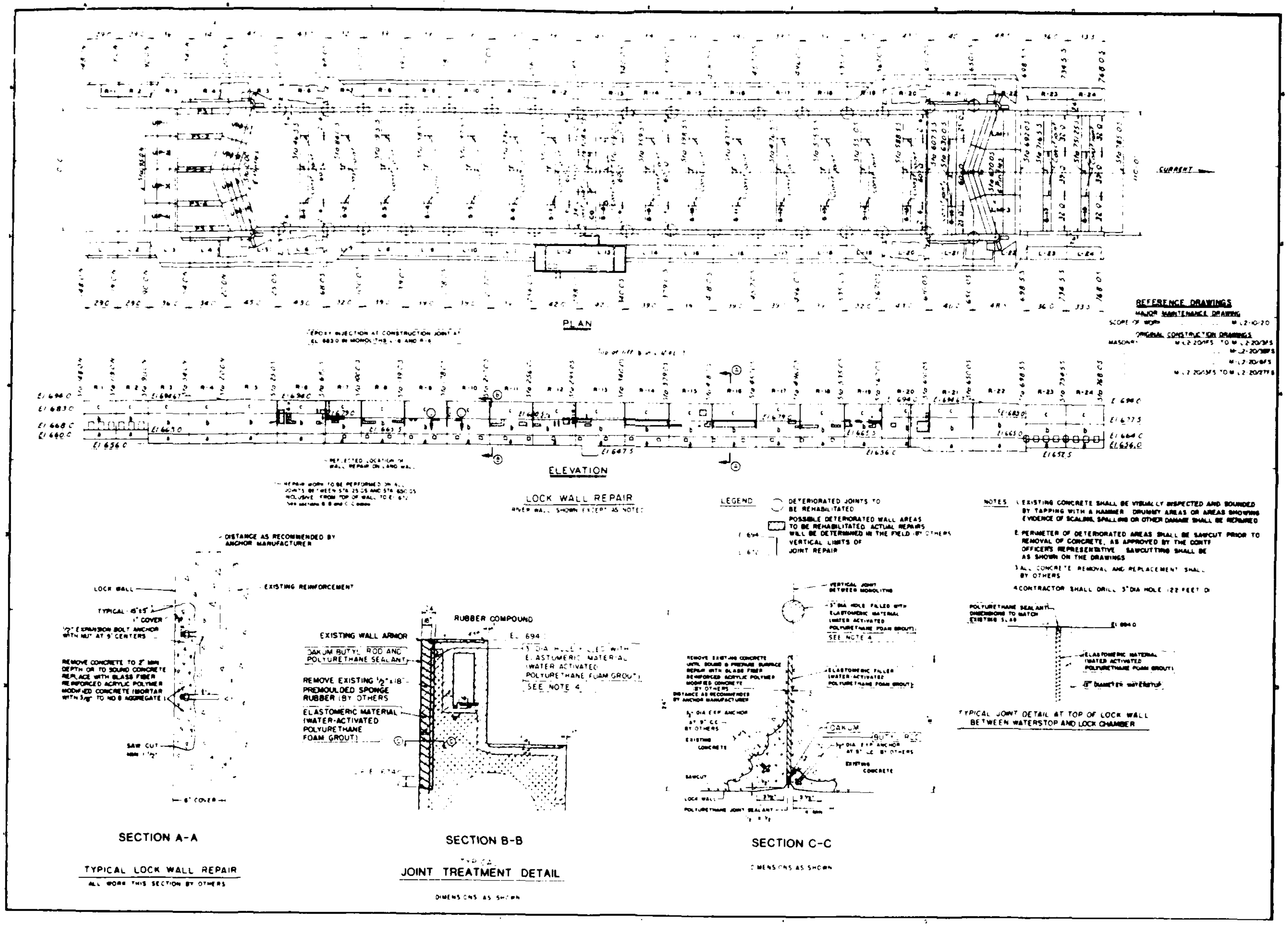


Figure 46. Monolith joint repair details, Lock and Dam No. 2

Figure 47. Monolith joint repair with bevel, Lock and Dam No. 2

Figure 48. Backing material and oakum in place before grouting, Lock and Dam No. 2

Figure 49. Pourirg water activated grout, Lock and Dam No. 2

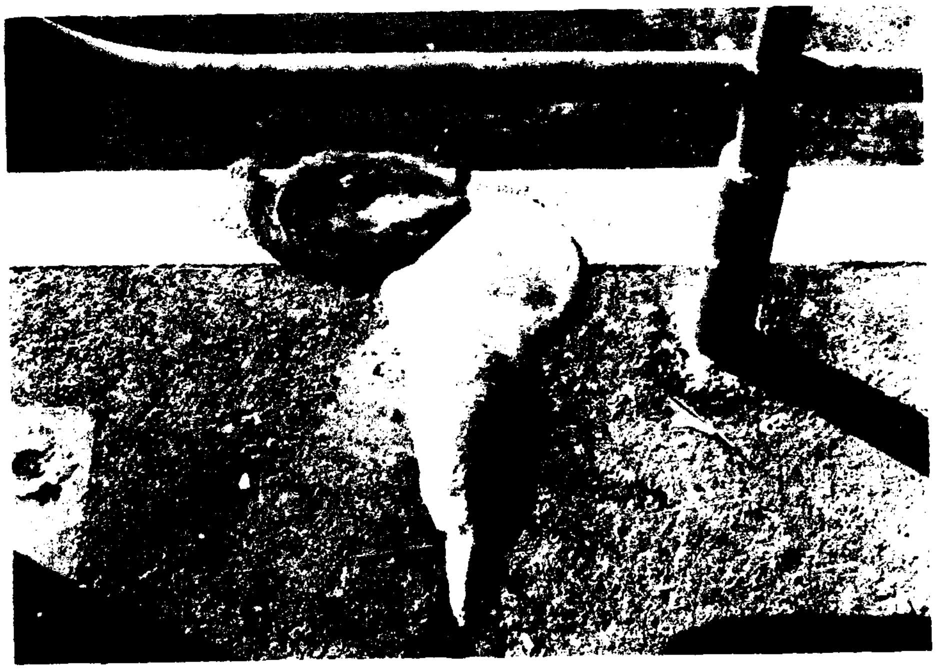


Figure 50. Expanded grout, Lock and Dam No. 2

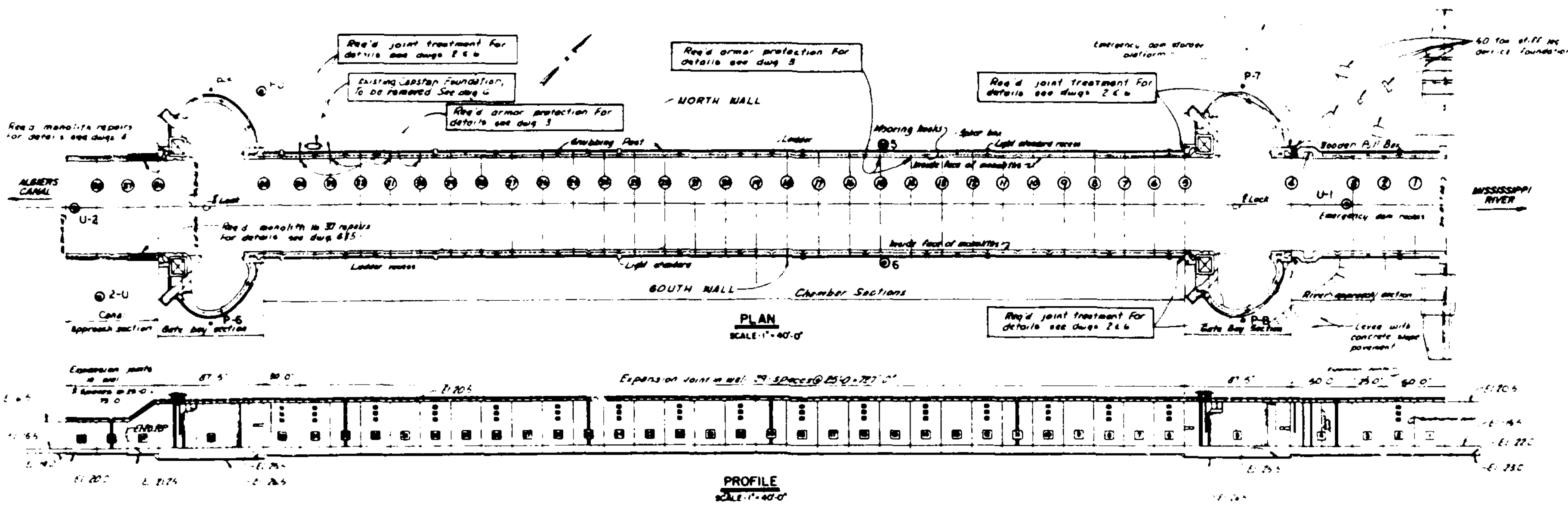


Figure 51. General plan, Algiers Lock

averaged between 50 and 55 ft in length. The lock was constructed from 1947 to 1952 with the concrete construction essentially completed in August 1950. The lock provides navigation between the Mississippi and an alternate route of the Gulf Intracoastal Waterway (McDonald 1986).

45. Periodic Inspection No. 5 (USAED, New Orleans, 1985a) indicates all of the corners of the monolith joints in the lock walls had been rounded-off, and some required immediate attention (Figure 52). The damage was attributed primarily to navigation-traffic impacts at the joints. The north wall joints were more damaged because the tows normally were tied to that wall. The joint damage ranged from 6 to 12 in. wide to 6 to 8 in. deep. Vertical rebars were exposed at several locations, and several large fractured pieces of concrete were on the verge of falling out. The report recommended repairs to the joints as soon as funding could be made available but in no case later than 2 to 3 years.

46. The District decided to repair the six most deteriorated joints: 14N, 15N, 30N, 31N, 33N, and 34N (Figure 51). The joint repairs were made by Professional Construction Services during the fall of 1988. The contract for $304,594 was awarded to repair the six joints and to repair leakage between the gate bay monoliths and adjacent monoliths. Underwater inspections had previously determined that the damage did not extend below the lower pool elevation; therefore, the test repairs were made without dewatering to minimize the disruption of normal operations. Two separate techniques were used to repair the joints. These repairs will be monitored for their performance until the next scheduled dewatering in 1993. The optimum repair technique will then be applied to all of the joints requiring repair as a part of a rehabilitation.

47. The first method of joint repair consisted of a modified version of the conventional armor system (Headquarters, Department of the Army, 1979). This system was used on joints 14N, 30N, and 33N. The second repair method consisted of the installation of an ultra-high-molecular weight polyethylene liner to protect the joint. This system was used on joints 15N, 31N, and 34N. Ultra-high-molecular weight polyethylene is a material with excellent impact and abrasion resistance and is not adversely affected by water, temperature changes, ultraviolet rays, or chemical action. It is not subject to corrosion, but if damaged can be easily replaced. This material is currently

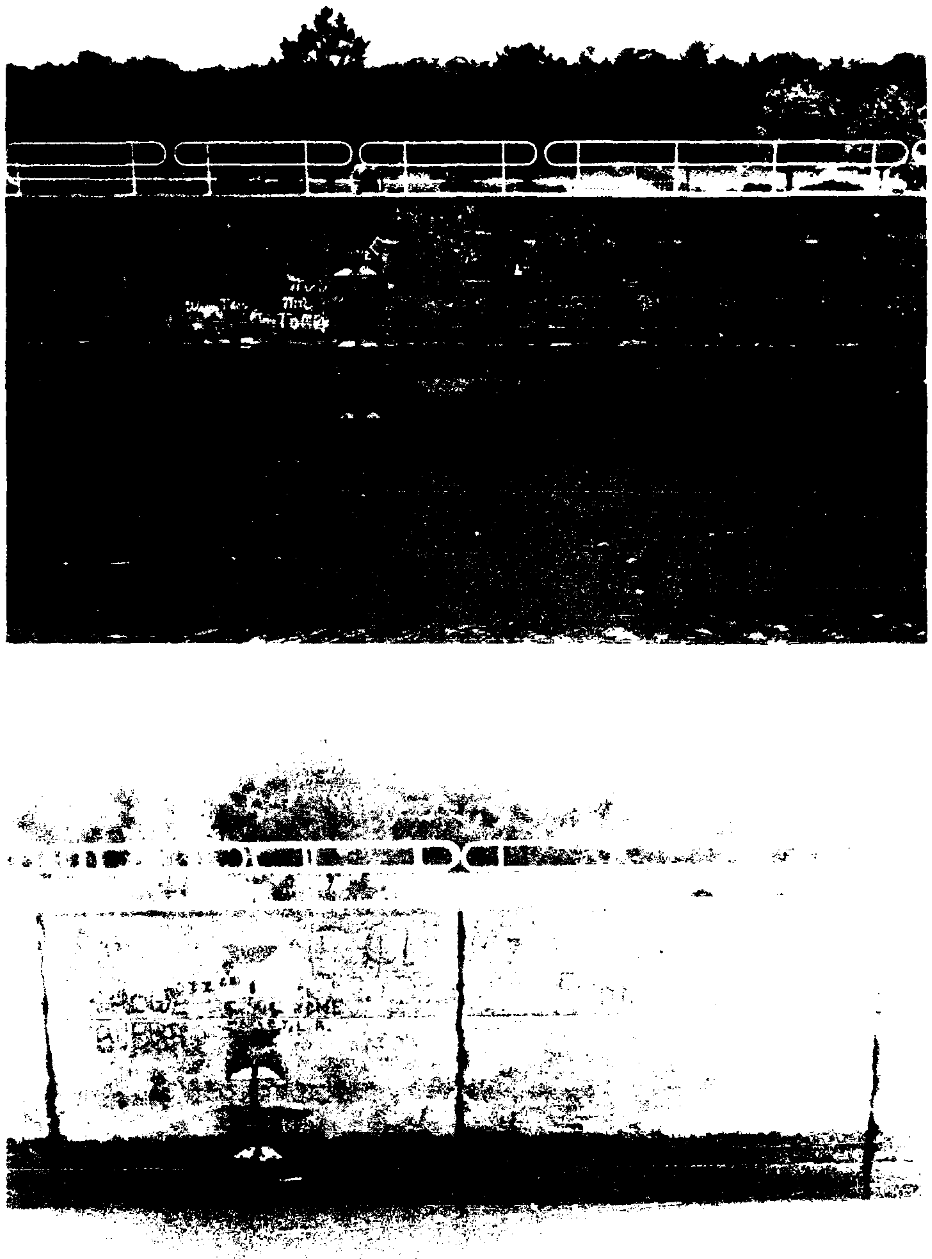


Figure 52. Typical examples of joint deterioration,
Algiers Lock

installed in the Port of Tacoma, WA, and in a dock facility in Corpus Christi, TX. No field data are available at this time. These methods were selected for their potential to protect the joint from future damage caused by impact.

48. The procedure for the first repair method began with the removal of the deteriorated concrete around the joint. The contractor used a system of scaffolding hung over the side of the lock as well as scaffolding mounted on a pontoon barge to gain access to the work area. A minimum 4-in.-deep saw cut was made 12 in. on each side of the joint. Concrete was removed a minimum depth of 8 in. with jackhammers. A final removal surface tolerance of plus or minus 1/2 in. was required. Two No. 6 dowels were installed on 12-in. vertical centers on both sides of the joint (Figure 53). One dowel had a standard 90-deg hook and the other had a standard 180-deg hook. All dowels were installed with an epoxy grout. Three No. 6 dowels were installed vertically the entire height of the repair. The concrete surface was prepared by wet sandblasting. No bonding agent was used. Next, the 1/2-in. preformed joint filler was installed. This step was followed by the installation of the 3/4-in.-thick armor with stud anchors along with the formwork. A 3-in. polyvinyl chloride tremie was used to place the new concrete in one continuous 20-ft lift. The concrete was designed to have a 0.45-water-cement ratio and a 90-day compressive strength of 3,000 psi. The 90-day strength requirement was based on the contractor's use of fly ash. After the initial strength had been developed, the formwork was stripped and the concrete was moist cured for 7 days. The armor was coated with a coal tar epoxy paint, and the joint sealant was installed.

49. The second repair technique used the same procedures for removing concrete and surface preparation. After the surface was prepared, the 1/2-in. preformed joint filler was installed. The ultra-high-molecular weight polyethylene liner was then coated with form-release oil and installed on the inside of the formwork. Simultaneously, the stainless steel studs which had been welded to the concrete anchor plates were placed on 12-in. vertical centers (Figure 54). Again a tremie was used to place the new concrete in one 20-ft lift. After the forms were stripped, the new concrete was moist cured for 7 days. The liner was fixed on one side and free on the other side to compensate for thermal expansion and contraction (Figure 55).

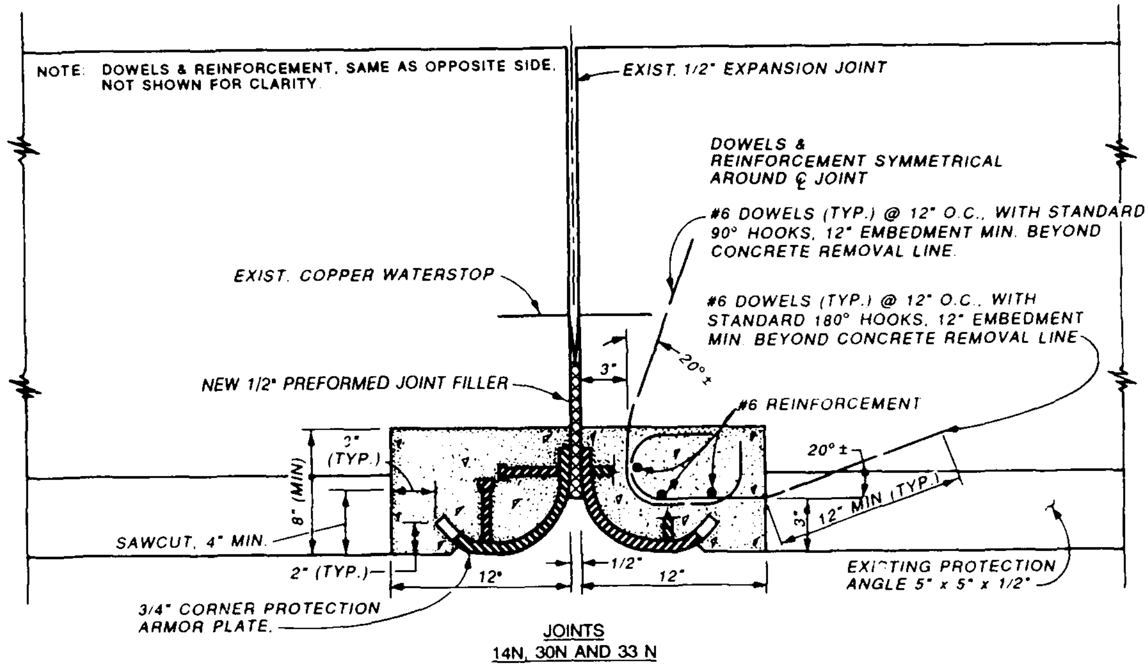


Figure 53. Monolith joint repair details for joints 14N, 30N, 33N, Algiers Lock

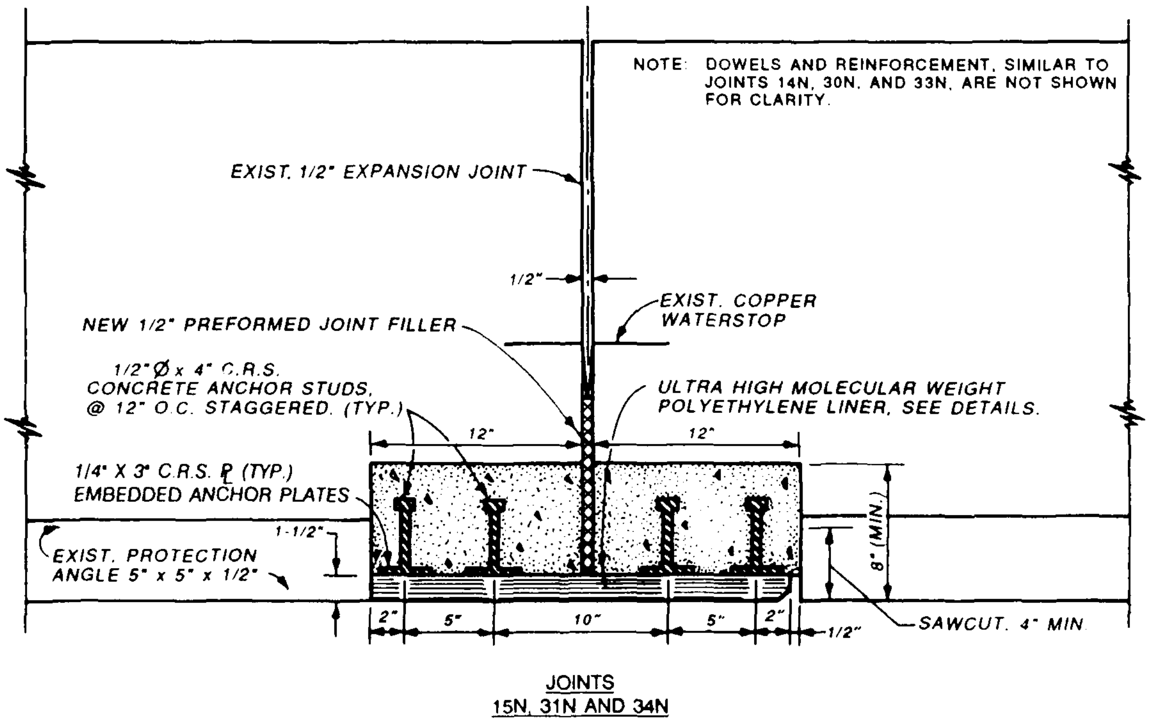
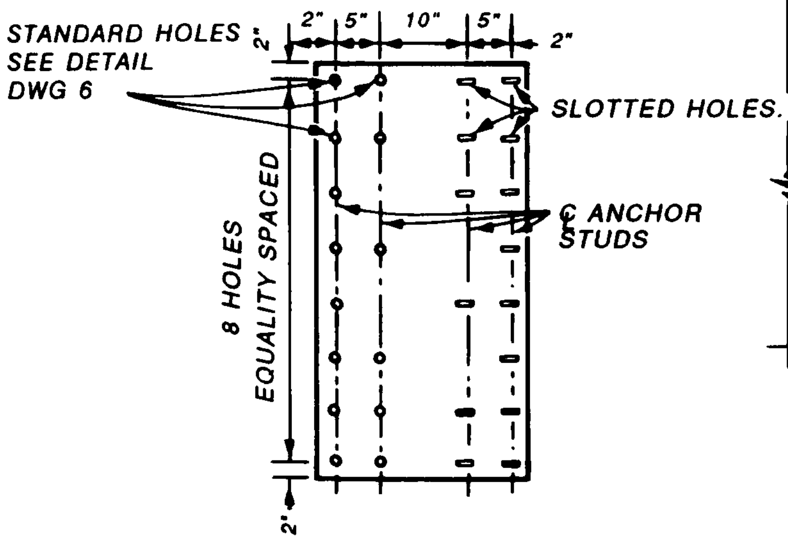
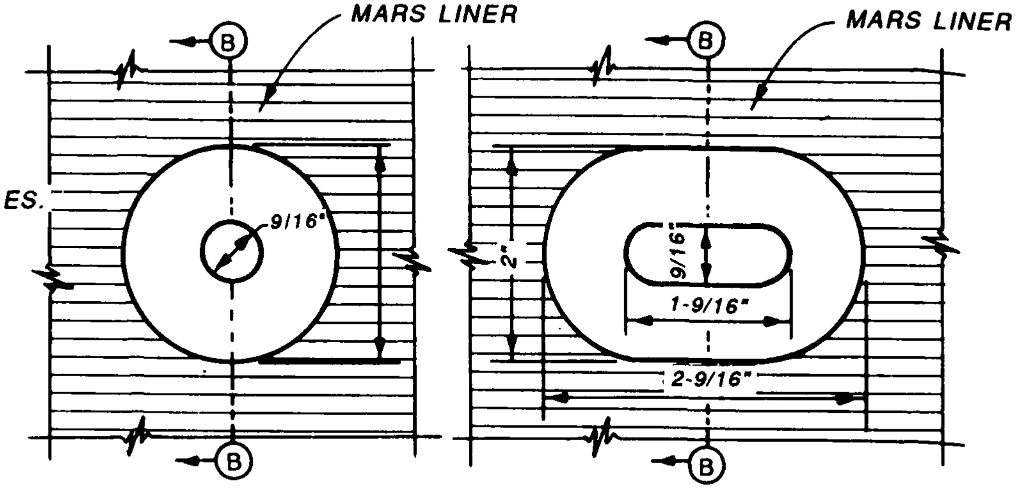


Figure 54. Monolith joint repair details for joints 15N, 31N, and 34N, Algiers Lock



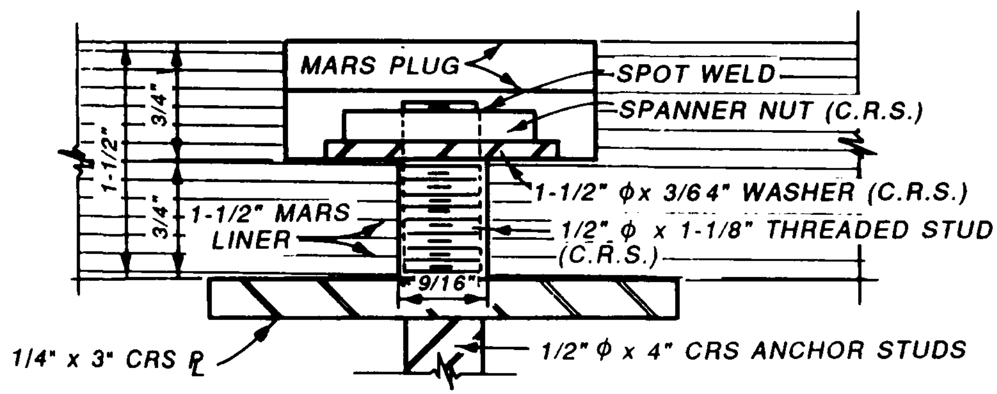
1-1/2" POLYETHYLENE LINER DETAIL

NOTES:
1. FOR LEGEND, SEE DWG. 2.
2. FOR GENERAL NOTES, CONCRETE NOTES AND STEEL NOTES, SEE DWG. 5.
3. SPACING OF #6 DOWELS AND CONCRETE ANCHOR STUDS ARE AT 12" O.C. TO ALLOW A 6" CLEARANCE BETWEEN SAME, FOR EASE OF INSTALLATION.
4. SEE NOTES DWG. 4.



MARS LINER
STANDARD HOLE

MARS LINER
SLOTTED HOLE



SECTION B

Figure 55. Ultra-high-molecular weight polyethylene liner details, Algiers Lock

50. These repairs have performed well so far and will be monitored for future performance.

Bayou Sorrel Lock

51. Bayou Sorrel is located in the East Atchafalaya Basin Protection Levee about 12 miles southwest of Plaquemine, LA. The structure consists of two reinforced concrete sector gates connected by a 600-ft-long earth chamber with timber guide walls in the lock chamber and on the upstream and downstream sides of the lock (Figure 56).

52. Periodic Inspection No. 4 (USAED, New Orleans, 1985b) indicated that all of the vertical joints experienced at least some degree of spalling. The monolith joints in the lock chamber walls were severely deteriorated, with reinforcing bars exposed, twisted, and missing (Figure 57).

53. The repair technique for the monolith joints consists of removing the deteriorated concrete a minimum depth of 8 in. (Figure 58) and then preparing the surface. The formwork would be installed with corner protection angles and Nelson stud anchors attached. New joint filler would be installed the length of the repair, and the repair material would be placed and cured.

54. These repairs are scheduled for the summer of 1989.

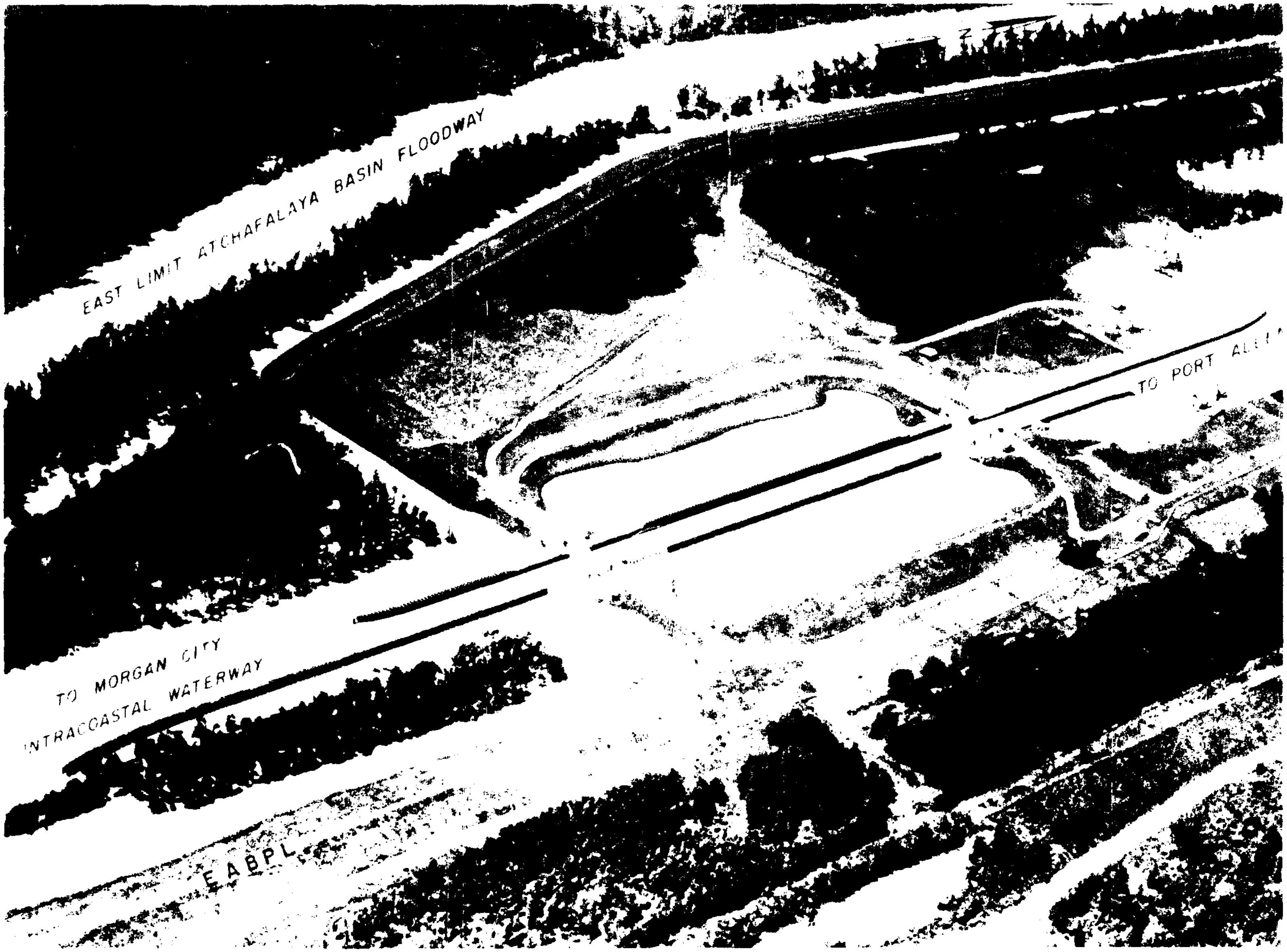


Figure 56. Aerial view, Bayou Sorrel Lock

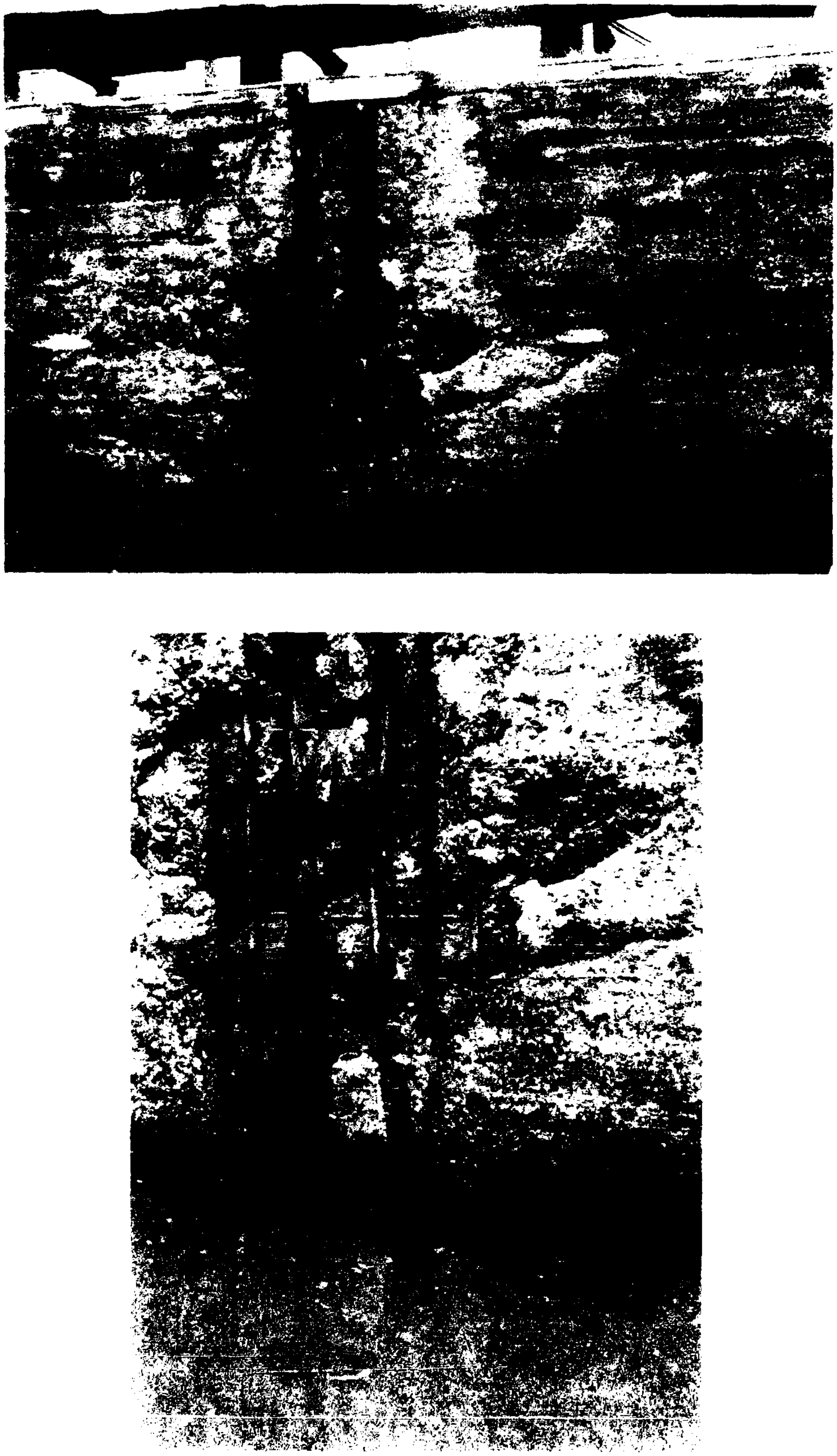


Figure 57. Extreme example of monolith joint deterioration, Bayou Sorrel Lock

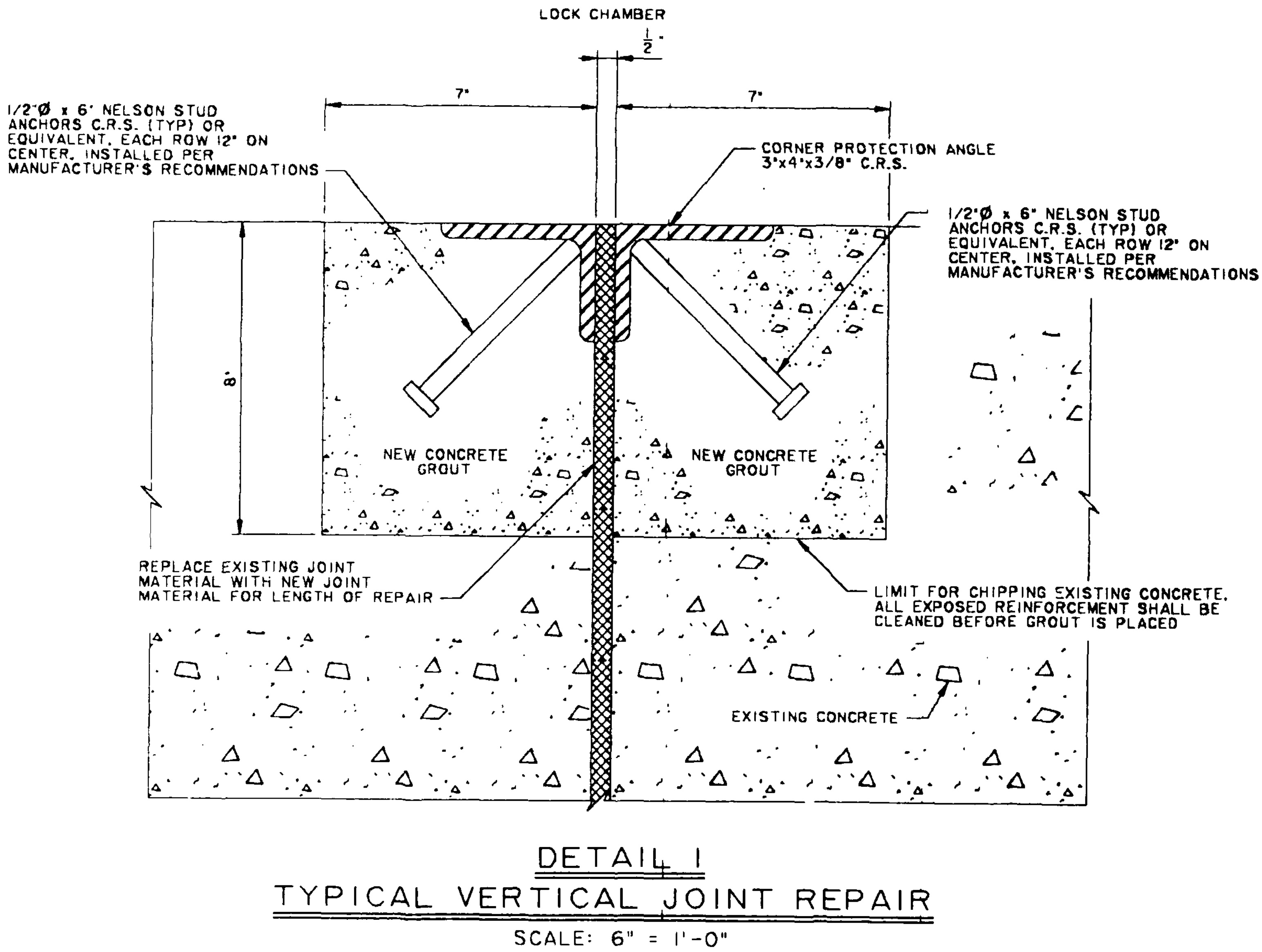


Figure 58. Monolith joint repair details (not final), Bayou Sorrel Lock

PART III: DISCUSSION

55. All of the repairs discussed in the case histories are performing well except for the Fibercrete and the Thorobond. The Fibercrete used on the test section of the upstream guide wall at Emsworth Locks and Dams exhibited impact failures, abrasion erosion, and delamination after only 3 months. An insufficient amount ot steel fibers (60 lb/cu yd as opposed to 200 lb/cu yd) could have contributed to this failure. The Thorobond repairs on the spillway monolith joints at Martis Creek Lake Dam have failed. This failure was probably the result of improper surface preparation and questionable material selection. Insufficient time has elapsed to compare the performance of the ultra-high-molecular weight polyethylene liner with that of the conventional steel armor at Algiers Lock.

PART IV: CONCLUSIONS AND RECOMMENDATIONS

Conclusions

56. Spalling is the most common deficiency for monolith joints in locks and dams when seepage and joint sealant failure are not considered (Table 1). Spalling is followed by distortion and cracking. Spalling is also the most common deficiency listed in the case history repairs.

Table 1

Types of Deficiencies at Monolith Joints

Deficiencies	Dams		Locks	
	No.	%	No.	%
Construction	1	0	0	0
Cracking	21	4	9	3
Disintegration	6	1	8	3
Distortion	67	11	22	8
Erosion	16	3	3	1
Joint Sealant	63	11	46	17
Seepage	218	37	77	29
Spalling	195	33	105	39

57. Although damage from impacts and cycles of freezing and thawing are the primary causes of deficiencies in the case history repairs, settlement is the primary cause listed in the WES damage and repair data base. For dams, settlement is followed by erosion, temperature, and maintenance faults. For locks, settlement is followed by weathering, shrinkage, and construction faults. No entries are listed under impact damage in the data base. It should be noted that the list of causes is not comprehensive because not all periodic inspection reports, the source of information for the data base, identify causes.

Repair Plan

58. The following areas, generally described in EM 1110-2-2002

(Headquarters, Department of the Army, 1986), must be addressed to make a successful monolith joint repair:

a. Evaluation of the joint.

b. Determination of the cause(s) of the deficiency.

c. Selection of the repair technique and material.

(1) Determining method of eliminating the cause(s).

(2) Determining constraints.

(3) Determining the options.

(4) Evaluating the options.

(5) Selecting the optimum solution.

d. Preparation of the design memoranda, plans, and specifications.

e. Execution of the plan.

59. Once the joint has been evaluated, the cause of the deterioration must be identified. The most common causes of monolith joint deterioration that require repair are damage from cycles of freezing and thawing, settlement, and impact. Damage from cycles of freezing and thawing occurs through direct action on the material itself and through the action of water, absorbed in the joint filler and freezing in the joint. For conventionally placed portland-cement repair materials, damage is mitigated through the use of entrained air. Some materials, such as dry-mix shotcrete, are not readily entrained with air. Shotcrete, however, is relatively impermeable and, therefore, not as likely as a conventional concrete material to become critically saturated. Wet-mix shotcrete, which can be air entrained, should be considered for repairs in areas subject to damage from cycles of freezing and thawing.* Acrylic polymer modified concrete is also relatively impermeable and has demonstrated an ability to resist damage from cycles of freezing and thawing. According to Technical Report REMR-CS-13 (McDonald 1987), repair sections thinner than the depth of frost penetration will not prevent damage to nonair-entrained concrete that continues to freeze when critically saturated. If settlement caused the damage, instrumentation records must be checked to determine if the settlement or differential movement is continuing. If

* If a repair material is to be used over nonair entrained concrete subject to saturation and freezing, ensure that the depth of the repair material is greater than the depth of frost penetration. If the depth of repair does not exceed the frost penetration depth, deterioration will likely occur in the nonair entrained concrete immediately behind the repair.

differential movement continues, the movement must be controlled, or the joint must be designed to accommodate it. Consolidation grouting to improve the foundation below a lock and posttensioned tendons have been used to minimize differential movement (Webster and Kukacka 1987). Differential movement of adjacent monolith joints causes damage directly from shearing and abrasion. Indirectly, damage is caused when a monolith extends beyond the adjacent monolith in the lock chamber and navigation traffic impacts the extended monolith. Joints can be protected from impact damage by being recessed and covered with a protective material. Steel armor plates or steel angles have been used traditionally, although new materials, such as the ultra-high-molecular weight polyethylene, are now available.

60. A wide variety of repair materials are available. Conventional concrete, fiber-reinforced concrete, wet- and dry-mix shotcrete, fiber-reinforced shotcrete, and epoxy grout are some of the materials that have been used in monolith joint repairs. In the selection of a repair material, several characteristics must be considered. The repair material should be compatible with the in situ material with emphasis on the coefficient of thermal expansion and modulus of elasticity. It should be strong and have the ability to resist damage from impact loads. The repair material should demonstrate volume stability during and after curing and develop good bond strength. The material must be able to perform in the wet lock-chamber environment and resist damage from cycles of freezing and thawing. Permeability and thermal stresses from the hydration process must also be considered, although they are not as critical for a joint repair as they are for a large-scale overlay. Finally, the economy of the material should be considered, both from the standpoint of initial cost and long-term maintenance cost.

61. Even the best material will not perform as intended if the proper procedures are not followed during construction. The area of deteriorated concrete must be identified and completely removed. Saw cuts should be used to prevent overbreakage and feathering of the repair. When the area of repair is relatively small, hand-held jackhammers and chipping hammers are often used. These two removal methods actually allow the operator to feel the difference between the deteriorated and sound concrete. The concrete surface to receive the repair material must be properly prepared to achieve a good bond. Generally, proper preparation means a clean, dry surface with a rough texture. Typically, sandblasting or high-pressure water-jet blasting is used. If

bonding agents are used, the manufacturer's instructions must be followed exactly, or a bond breaker can result. Previous WES studies have demonstrated that a majority of the bond strength is a result of good bond between the repair material and a properly prepared surface. Bonding agents can at best provide only a marginal increase in bond strength. Anchor bolts also are used to connect the repair material and provide protection from brittle failures (Liu and Holland 1981). Because the working area is generally restricted to a tall, narrow shaft once the form work is in place, special attention must be made to ensure that the repair material is properly placed and consolidated. Proper curing should begin as soon as possible to ensure a strong repair with minimal cracking. Repairs normally are made during regular lock downtimes which are scheduled to have minimum impact on the transportation industry. In the North, this downtime will generally be during the coldest part of the year. For this reason repairs frequently require weather proofing plans. Before, during, and after batching and placing, materials must be protected from the weather.

Recommendations

62. Field repairs should continue to be monitored and evaluated for effectiveness. This information should be presented in The REMR Notebook.*

63. The periodic inspection report is the primary source of information regarding the status of Corps structures. ER 1110-2-100 (Headquarters, Department of the Army, 1983), which gives guidance on the preparation of periodic inspection reports, requires that an examination be conducted in accordance with EM 1110-2-2002 (Headquarters, Department of the Army, 1986), when deterioration is imminent or has already occurred. EM 1110-2-2002 calls for a concrete repair report to be included in the periodic inspection report at the conclusion of any unique concrete repair. Including this detailed information in all of the periodic inspection reports would be extremely beneficial in determining the optimum strategy for future repairs.

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* A publication issued as part of the effort under Work Unit 32282, "Program Management," of the REMR Research Program. Notebook is updated as necessary by supplements, corrections, and revisions in loose-leaf form.

64. A variety of techniques have been used to successfully repair monolith joints. Presently there is a limited understanding of what constitutes the optimum repair technique with respect to cost, constructability, and durability. Additional research is necessary to determine the optimum repair technique.

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